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Investigating Use of Aggregate Density to Develop Design Aggregate Structure for Asphalt Concrete Mixtures

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By

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ABSTRACT

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In recent years there have been several instances of premature failures of pavements in West Virginia due to rutting. While there are various potential causes for the failures, aggregate density within the mix when designed at 80 gyrations was of specific concern to this thesis and was compared to the maximum dry density of the aggregate as measured using ASTM D4253-00. The objective of this research was to determine if the aggregate in the asphalt concrete mixture was reaching a dense configuration.

The Bailey Method was used to choose gradations and the selected research methodology supported the evaluation of the Bailey Method as a means for estimating changes in mix volumetric properties with changes to aggregate gradation. Aggregate density at various locking point definitions, as well as, N_{des} and N_{max} of 80 and 125 gyrations respectively, were evaluated to assess the effects of compacting to current N_{des} level versus locking point.

Statistical methods were used to analyze the various test results including, line of equality charts and t-tests to test for equality. All results were evaluated at a 95% confidence level for consistency. It was determined that the aggregates in the asphalt concrete specimens achieved a dense aggregate structure when compacted to 80 gyrations when compared to the dry density specimens. Also, as the CA ratio increases the IDT strength increases indicating more resistance to rutting in the field. This could be a useful tool in choosing a gradation as the CA ratio can be determined from the gradation.

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TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

Chapter 1 INTRODUCTION

1.1 Background

Since the introduction of the Superpave mix design method for asphalt concrete in 1994, design level of gyrations, N_{des}, has been an area that has been evaluated and refined a number of times. Original recommendations included 28 levels for N_{des} based on traffic levels and climactic regions. NCHRP Project 9-9 studied and evaluated these recommendations and refined them to only 4 N_{des} levels based on traffic. In 1999 another study was carried out by the Federal Highway Administration and the N_{des} values were refined further and eventually adopted by AASHTO in 2001 (Prowell and Brown, 2007). West Virginia MP 401.02.28, which is a guide to designing hot-mix asphalt using Superpave, requires lower values for N_{des}, using80 gyrations for 3 to 30 million ESALs versus the 100 gyrations specified in AASHTOR35.

Shortly after the change in the WVDOH Superpave mix design MP, two pavements developed bleeding and rutting. In both cases a forensic evaluation of the mix designs demonstrated the mixes were in compliance with the WVDOH MP. Although many successful Superpave mixes were designed and placed using the new MP, the two failures raised concern that there was a potential problem that should be investigated.

The goal of the reduction in the compaction effort during mix design was to increase the design binder content of the mixes. However, other than the control points for aggregate gradation, there is no specific guidance in the Superpave mix design method for the selection of a design aggregate structure. Furthermore, the Superpave mix design method is based solely on volumetric analysis. There is no test required to evaluate mechanical properties of the mix. Due to the two failed pavements there was a concern that the reduction in compactive effort resulted in an aggregate structure that was not in a dense configuration resulting in a pavement that was susceptible to bleeding and rutting.

1.2 Problem Statement

The permanent deformation of asphalt concrete, known as rutting, is a failure in which longitudinal depressions develop in the wheel paths of a roadway under repeated traffic loading. While there are multiple causes for this failure, it is a goal of this thesis to evaluate the interaction between aggregate density in the mix design versus maximum aggregate density.

1.3 Research Objectives

The objective of this research was to evaluate methods for determining if mix designs prepared under the current WVDOH specification produce a design aggregate structure with a dense configuration that promotes stability of the mix. The selected research methodology also supports an evaluation of the Bailey Method as a means for estimating how changes in aggregate gradation affect volumetric properties.

1.4 Scope and Limitations

This thesis evaluated aggregate from two West Virginia suppliers, Greer Limestone and Jefferson Asphalt. All asphalt tests were performed with equipment available in the West Virginia University Asphalt Technology Laboratory. The maximum dry density tests were carried out with the standard testing mold specified in ASTM D4253-00 and the vibratory table located in the West Virginia University Concrete Laboratory. The Bailey Method spreadsheet developed by Zaniewski and Mason (2006) was used to choose mix gradations in accordance with the Bailey Method principles.

The maximum density of the aggregate was evaluated for comparison to the density of the aggregate in the mixes. The Bailey Method provides guidance for predicting the proper gradation for a specific blend of aggregate to achieve coarse aggregate interlock, proper aggregate packing and volumetric requirements (Vavrik, 2000). This method was used to design multiple gradations that meet all of the Superpave mix design requirements. The mixes were evaluated using the locking point concept (Vavrik, 2000) and the indirect tensile strength, IDT, was measured.

Chapter 2 LITERATURE REVIEW

2.1 Introduction

A review of rutting in asphalt pavements and how aggregate gradation can impact the resistance of a pavement to rutting explains the importance of this topic. In addition to discussing the topics included in this thesis, the literature review was used to help develop the framework for the experimental process. Areas of interest include reviewing the Superpave mix design process, maximum dry density of the aggregates, locking point and the Bailey method for use when making adjustments to aggregate gradations.

2.2 Aggregate Gradation

Aggregate gradation is the distribution of particle size of a stockpile or blend of aggregate, typically expressed as the percent of total weight passing each sieve. The Marshall and Superpave mix design methods provide guidance on aggregate gradation by specifying control points for selected sieves for each mix type, Table 1(WV MP401.02.28). These points specify a range of percent passing specific sieves that should not be exceeded for a mix design. The Superpave mix types are defined by the nominal maximum aggregate size, NMAS, which is one sieve size large than the first sieve to retain 10 percent or more of the aggregate blend.

In addition to the control points, Superpave identifies primary control sieves, PCS, and a corresponding percent passing for each mix type as given in Table 2 (AASHTO M 323). The percent passing establishes whether a mix is coarse or fine. A mix with a percent passing the PCS greater than the specified value is considered a fine mix. A coarse mix has a percent passing the PCS less than the specified value.

Table 1: Superpave mix gradation control points

Type of Mix	37.5	25	19	12.5	9.5	4.75
	Nominal Maximum Size					
Standard Sieve Size	37.5 mm (1 1/2 in.)	25 mm (1 in.)	19 mm (3/4 in.)	12.5 mm (1/2 in.)	9.5 mm (3/8 in.)	4.75 mm (No. 4)
50 mm (2")	100					
37.5 (1 1/2")	90 to 100	100				
25 mm (1")	90 max	90 to 100	100			
19 mm (3/4")		90 max	90 to 100	100		
12.5 mm (1/2")			90 max	90 to 100	100	100
9.5 mm (3/8")				90 max	90 to 100	95 to 100
4.75 mm (No.4)			*		90 max	90 to 100
2.36 mm (No.8)	15 to 41	19 to 45	23 to 49	28 to 58	32 to 67	
1.18 mm (No.16)						30 to 60
0.6 mm (No.30)						
0.3 mm (No.50)						
0.075 mm (No.200)	0.0 to 6.0	1.0 to 7.0	2.0 to 8.0	2.0 to 10.0	2.0 to 10.0	6.0 to 12.0

*When using 19 mm mix for heavy duty surface mix additional requirement of minimum 47% passing 4.75 mm sieve. Allowable tolerance of JMF±5% on the 4.75 mm sieve, but must be above the minimum limit.

Table 2: Primary control sieves

Type of Mix	PCS	Percent Passing PCS
37.5	9.5	47
25	4.75	40
19	4.75	47
12.5	2.36	39
9.5	2.36	47

2.3 Fuller Curve and Maximum Density

Brown et. al. (2009) states that around 1907 Fuller and Thompson determined that the maximum density of aggregates can be estimated as:

$$\boldsymbol{P_i = 100(\frac{d_i}{D})^n} \qquad \text{Equation 1}$$

Where: P_i = total percent passing the sieve n = 0.5

d_i = diameter of sieve in question

D = maximum aggregate size

Research by the FHWA in the 1960s refined the exponent to 0.45. Aggregate charts are plotted with the percent passing on the ordinate and the sieve size on the abscissa. The abscissa is scaled using the 0.45 power. The maximum density line is drawn from the origin (0,0) to 100% passing and the maximum aggregate size. The gradation limits for each Superpave mix type are established by control points as shown in Table 1(Federal Highway Administration, 1988).

The Fuller Curve indicates the gradation that will produce the densest configuration when mixed at these proportions. However, every gradation regardless of where it lies on the size distribution chart has a maximum and minimum achievable density which can be determined by ASTM D4253-00 and ASTM D4254-00 respectively. These standards are written for use with cohesionless soils, however, it was determined that the aggregate blends used in this thesis met all the specifications listed in the ASTM D4253-00 and ASTM D4254-00 and therefore are used. The test method limits use of soils where no more than 15 percent pass a No. 200 sieve and 100 percent must pass a 3 inch sieve.

The basic process for obtaining maximum density involves filling the mold flush to the top with the sample and applying weight to achieve a surcharge of 2 psi. The apparatus is placed on a vertically vibrating table for a specified amount of time depending on the frequency of vibration. The maximum density is calculated as:

$$\boldsymbol{\rho_{max} = \frac{M_s}{V}}$$ Equation 2

Where: ρ_{max} = maximum density on sample

M_s = mass of solids

V = volume of mold occupied to top of sample

The significance of this test is that it would allow the comparison of the maximum density of the dry aggregate to the density of the aggregate in the pills. This would determine if the mixtures are in fact achieving a dense aggregate configuration.

2.4 Mix Design Methods

Mix design in the United States has been around since the late 1860s when N.B. Abbot used tar as a binder for a bituminous pavement. These early pavements did not perform well as surface mixes and the tar was blamed due to interests in Trinidad Lake Asphalt (Crawford, 1989). However, it was evident that during the early years, the importance of aggregate gradation was not yet understood and poor gradations may have been the factor that doomed these pavements to failure (Crawford, 1989). In 1870 Edmund J. DeSmedt obtained a patent and placed the first asphalt pavement in the United States in Newark, NJ ("History of Asphalt," 2013). In the early 1900s the Warren Brothers obtained patents for Bitulithic pavements and started developing large stone mixes using 19 mm and 32 mm maximum aggregate sizes (Brown, et. al., 2009). As time and technology progressed various mix design methods were developed to attempt to produce better performing asphalt pavements including the Hveem, Marshall and Superpave methods. Today in West Virginia the Marshall and Superpave mix design methods are the only ones used, therefore, the Hveem will not be discussed further. Also, since the investigation of design aggregate structure and locking point is a focus of this research, and the locking point is defined based on sample heights and gyration levels in the Superpave gyratory compactor, SGC, the Marshall method will not be discussed either.

2.5 Superpave Mix Design Method

The Superpave mix design method is the most recent development in asphalt concrete mix design methodologies. It was the culmination of a five year cooperative effort by the Strategic Highway Research Program, SHRP, to develop a rational mix design method. The basic steps involved in the Superpave mix design method are listed below (WV MP 401.02.28):

1. Selection of materials to be used. This includes asphalt binder, aggregate and reclaimed asphalt concrete, RAP, if used. Selection of asphalt binder is based on pavement temperatures over the previous 20 years, as well as, predicted traffic levels and speed. Aggregate specifications include consensus properties such as coarse aggregate angularity and fine aggregate angularity as found by AASHTO T 326 and AASHTO T 304 respectively. Additional consensus properties are flat and elongated particles (ASTM D4791) and sand equivalency test (AASHTO T 176 or ASTM D2419).There are additionally source properties that must be met for each stockpile including toughness, soundness and deleterious materials. These aggregate specifications were not part of the initial work on developing Superpave but were added later by a group with expertise in aggregates (Brown, et. al., 2009).
2. Design aggregate structure. When the Superpave method was introduced there was a concern that mix design technologists needed a process for selecting a suitable design aggregate structure. This process involves choosing at least three aggregate gradations to be blended from the stockpiles chosen in the previous

step. A trial binder content is determined for each gradation and two specimens, or pills, measuring 115±5 mm high by 150 mm in diameter are compacted in the Superpave gyratory compactor, SGC to determine bulk specific gravity, G_{mb}. The samples are compacted using a specified number of gyrations as described in the following section. In addition to the compacted pills, two samples are needed to determine the maximum theoretical specific gravity, G_{mm}, of the mix. The specimens are evaluated using the standard Superpave volumetric analysis equations, AASHTO R 35-09, and a gradation is chosen along with adjusted binder content to achieve 4% air voids in the total mix, VTM. The developers of the Superpave method recognized that as experience was gained with the method they would be able to use their experience to select a suitable design aggregate structure, DAS.

3. Design binder content. Both G_{mm} and G_{mb} specimens are created using the selected gradation and 4 binder percents; the estimated binder content, estimated binder content ±0.5%, and +1.0%. Volumetric calculations are again performed on these samples and the results are plotted against the percent binder content. Design binder content is chosen from the VTM graph at 4% air voids. The other volumetrics: VMA, voids filled with asphalt, VFA, theoretical maximum specific gravity of the mix, G_{mm} at N_{ini}, initial compaction level for the mix, and dust-to-binder ratio, D/B, are selected and compared to the mix design criteria. If all of these parameters meet the requirements for Superpave mix design criteria then the mix is acceptable. Table 3 shows the Superpave mix design criteria taken from WV MP 401.02.28. The VMA minimum requirements in Table 3 are 0.5 percent

higher than the requirements in AASHTO R 35-09. The WVDOH implemented this change concurrently with the reduction in compaction effort. Both the VMA and compaction effort changes were implemented to increase the design binder content as compared to Superpave mix designs prepared under the previous MP requirements. If these criteria are not met, three new DAS blends must be determined and the process must be repeated.

4. Evaluation of moisture sensitivity. This procedure is conducted in accordance with AASHTO T 238 in which six specimens are compacted to 7.0 ± 0.5% air voids at the design aggregate structure and design binder content. Three of the specimens are deemed control and are set aside while the other three are conditioned with vacuum saturation and freeze thaw cycles. All six samples are tested with the Marshall loading frame for their indirect tensile strength, IDT. The moisture susceptibility is determined by the average of the IDT strength of the conditioned samples divided by the average of the IDT strength of the control samples. A minimum ratio of 80% must be achieved for an acceptable mix (WV MP 401.02.28).

Table 3: Superpave mix design criteria

VTM						4%
Dust-to-Binder Ratio*						0.6-1.2
Tensile Strength Ratio						80% min
Minimum VMA (%)	Nominal Maximum Aggregate Size (mm)					
	37.5	25	19	12.5	9.5	4.75
	11.5	12.5	13.5	14.5	15.5	16.5
% VFA	65-75	68-76	70-78	72-79	74-80	75-81

*For coarse mixes D/B ratio range is 0.8 – 1.6

2.6 Superpave Compaction Effort

Since the implementation of the Superpave mix design method changes have been made to attempt to improve the results of asphalt pavements designed with this method: specific to this thesis, adjustments to N_{des}, number of gyrations for mix design. Originally, there were 28 recommendations for N_{des} gyrations based on 7 traffic levels and 4 climate regions across the United States. In an effort to refine these recommendations, NCHRP Project 9-9 was conducted and in 1999, 4 N_{des} values were proposed based on traffic levels. AASHTO adopted the recommendations in 2001 and they continue to be used today (Prowell and Brown, 2007). Prowell and Browns' (2007) NCHRP Report 573 evaluated gyration levels based on laboratory and field mix density and resulted in recommendations for reduction in gyration levels to improve pavement performance. The recommendations were also separated for binders with high temperature ratings of less than PG 76 and greater than or equal to PG 76 based on analysis of the shear stiffness of the binder (Prowell and Brown, 2007).West Virginia DOH adopted the N_{des} gyration levels recommended in NCHRP 573 and the compaction criteria for AASHTO R 35 and WV MP 401.02.28 can be found in Table 4 and Table 5 respectively.

Table 4: AASHTO R 35 Superpave gyratory compaction parameters

	Compaction Parameters		
Design ESALs (million)	N_{ini}	N_{des}	N_{max}
< 0.3	6	50	75
0.3 to < 3	7	75	115
3 to < 30	8	100	160
≥ 30	9	125	205

Table 5: WV MP 401.02.28 Superpave gyratory compaction parameters

	Compaction Parameters	
	Gyration Level-1	Gyration Level-2
20 Year Projected design ESALs (millions)	N_{des} for Binder < PG 76-XX	N_{des} for Binder ≥ PG 76-XX or Mixes Placed Below Top Two Lifts
< 0.3	50	50
0.3 to < 3	65	65
3 to < 30	80	65
≥ 30	100	80

2.7 Bailey Method of Gradation Analysis

Due to the trial and error nature of the design aggregate structure process, it is apparent that there is a need for a more systematic approach. The Bailey Method for gradation analysis is one system that relates gradation parameters with VMA to predict changes in volumetrics with adjustments to gradation. In an attempt to use a more scientific method to make adjustments to gradations, this thesis used the Bailey Method discussed below as part of the testing procedure.

The Bailey Method, originally developed by Robert Bailey with the Illinois Department of Transportation, is a systematic approach to choosing or adjusting aggregate gradation for asphalt concrete applications. The system was further refined by William Vavrik and Bill Pine to allow use with any dense-graded asphalt and stone mastic mixtures. The basic concept of the Bailey Method states that an asphalt concrete mixture get its strength and rut resistance characteristics from a strong aggregate skeleton formed by the coarse aggregate. Durability of the mix is achieved by adjusting the coarse and fine fractions to allow sufficient voids in the mineral aggregate, VMA, resulting in adequate amount of asphalt binder in the mix (Vavrik, et. al., 2002). Vavrik (2000) used the Bailey Method as the basis for his early research into aggregate gradation and VMA predictions. Additionally, Zaniewski and Mason (2006) studied the predictive abilities of the Bailey Method on VMA for West Virginia Superpave and Marshall Wearing mixtures. The outcome of their study showed promising results for the Superpave mixtures while yielding less conclusive results for the Marshall mixes. The Marshall Wearing I fine mix gave the most irregular results with the VMA actually decreasing when the Bailey analysis predicted an increase. They determined that since the Marshall mixes used natural sands while Superpave mixes use all crushed limestone, this contributed to the irregular results. Natural sands have a smooth and rounded texture which could cause significant changes in the Bailey predicted VMA values since it does not directly account for aggregate shape characteristics.

2.7.1 Aggregate Packing Principles

The Bailey Method uses aggregate packing principles as the foundation for building a strong aggregate skeleton in the asphalt mixture. Aggregate packing is typically constrained to analysis of the packing of spheres (3-D) or circles (2-D). In the analysis of 2-dimensional shapes, four conditions are considered (Vavrik, et. al. 2002):

- All round particles with void size 0.15d
- Two round, one flat particles with void size 0.20d
- One round, two flat particles with void size 0.24d
- Three flat particles with void size 0.29d

where d = diameter of nominal maximum particle size, NMPS

Figure 1 shows a visual description of these conditions as described in a Bailey Method workshop by Bill Pine (Pine, 2004). The conclusion of this 2-dimensional analysis uses the average of these four situations, 0.22d, to determine the most accurate prediction for the average aggregate size ratio for asphalt mixtures. In other words, this is the ratio of the diameter of the coarse aggregate that will create the voids, to the fine aggregate that will perfectly fill the void created. This ratio is one of the main concepts that the Bailey Method relies on as described later.

Similar results have been obtained when performing 3-dimensional analysis. Reed (1998) states that spheres of uniform particle size can be arranged into 5 different packing arrangements: cubical, orthorhombic, tetragonal, pyramidal and tetrahedral. When analyzing the size of a sphere that would fit into the voids created by these arrangements, the ratios ranged from 0.15 for tetrahedral packing to 0.42 for simple cubical packing of spheres (Reed, 1998). It was decided that the particle size ratio of 0.22, as used in the 2-dimensional analysis, would be appropriate in 3-dimension as well since the packing of aggregate is somewhere between cubical and tetrahedral but more toward tetrahedral arrangement. Figure 2 gives a visual of these two arrangements (Vavrik, et. al., 2002).

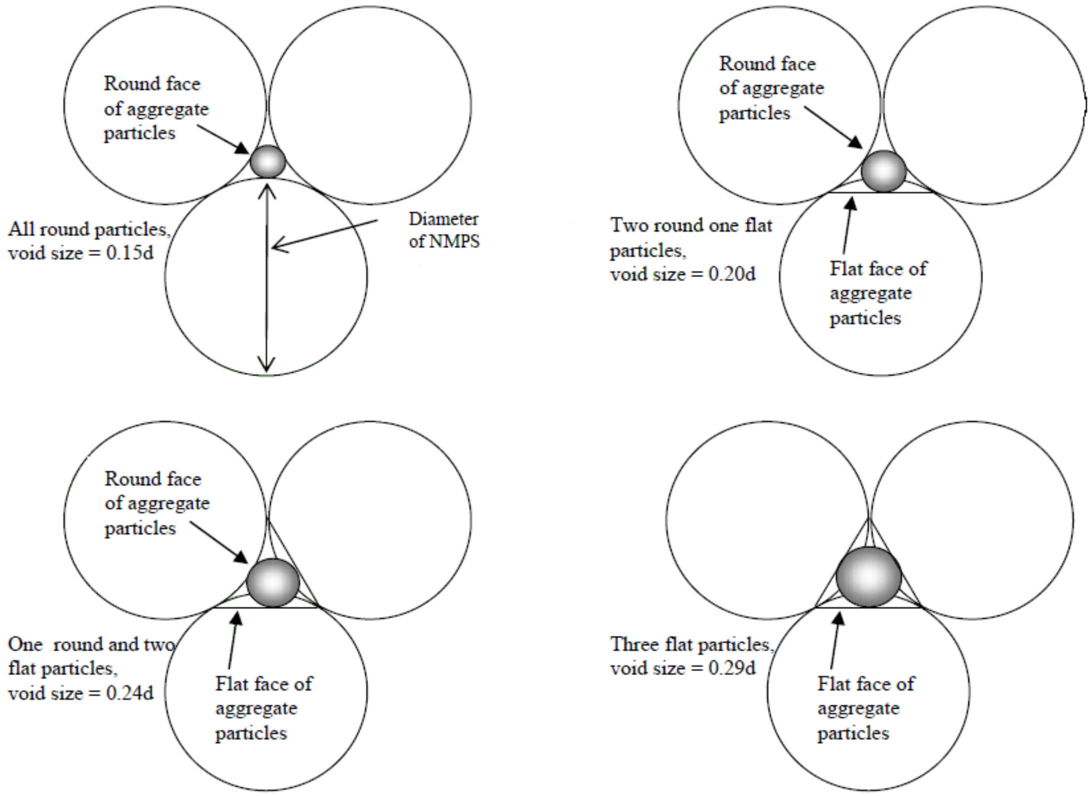

Figure 1: Void size estimations

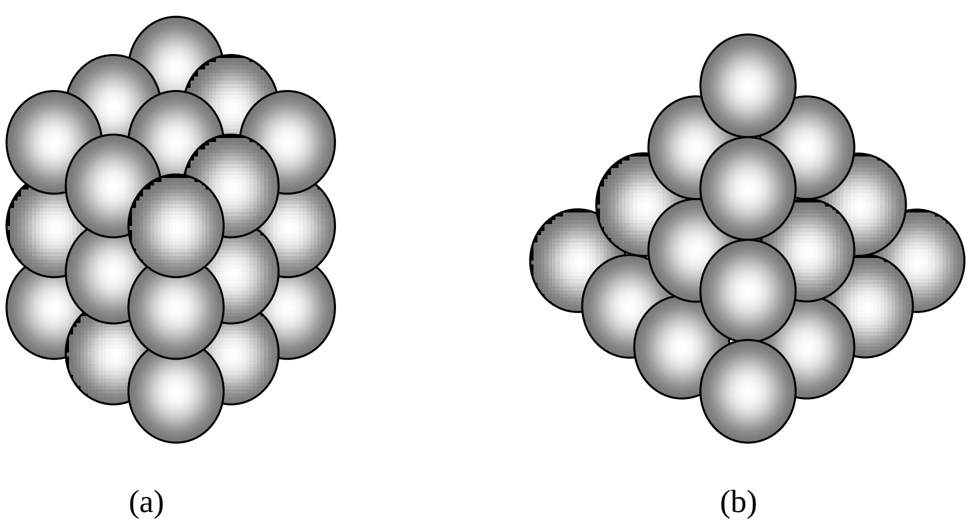

Figure 2: 3-dimensional packing of spheres (a) cubical (b) tetrahedral

This particle size ratio is one of the main principles used in the Bailey Method for defining coarse and fine fractions. (The Bailey definition of coarse and fine fractions is dependent on aggregate size. The Bailey Method does not use the common 4.75 mm sieve to separate coarse and fine aggregates). While analysis of spheres does not provide an exact visualization of aggregate packing it has been used successfully for a number of years and therefore continues to be the method of choice (Vavrik, et. al., 2002).

2.7.2 Bailey Principles

The Bailey Method focuses mainly on aggregate packing and the relationship between the coarse and fine fractions in the gradation. By following the guidelines and practices laid out in the Bailey Method, the designer is able to select and adjust the coarse aggregate fraction to achieve a strong aggregate skeleton to be more resistant to permanent deformation. Additionally, the fine aggregate fractions can be adjusted to allow enough VMA for sufficient binder to ensure a more durable mix. The Bailey Method uses 14 parameters for the evaluation of all mixes. For mixes where the fine aggregates control the properties, an additional 7 “new” parameters are used as listed in Table 6.

Table 6: Definition of Bailey terms

Bailey Term	Description
CA	Coarse Aggregate
FA	Fine Aggregate
SMA	Stone Matrix Asphalt
NMPS	Nominal Maximum Particle Size
PCS	Primary Control Sieve
SCS	Secondary Control Sieve
TCS	Tertiary Control Sieve
HS	Half Sieve
LUW	Loose Unit Weight
RUW	Rodded Unit Weight
CUW	Chosen Unit Weight
CA Ratio	Coarse Aggregate Ratio
FA_c Ratio	Fine Aggregate Coarse Ratio
FA_f Ratio	Fine Aggregate Fine Ratio
New NMPS	Nominal Maximum Particle Size for Fine Mixes
New PCS	Primary Control Sieve for Fine Mixes
New SCS	Secondary Control Sieve for Fine Mixes
New TCS	Tertiary Control Sieve for Fine Mixes
New HS	Half Sieve for Fine Mixes
New CA Ratio	Coarse Aggregate Ratio for Fine Mixes
New FA_c Ratio	Fine Aggregate Coarse Ratio for Fine Mixes
New FA_f Ratio	Fine Aggregate Fine Ratio for Fine Mixes

The Bailey Method combines coarse and fine fractions by volume as opposed to the standard practice of blending aggregate by weight. To do this, the loose and rodded unit weights, LUW and RUW respectively, are determined. Coarse and fine LUW and RUW are determined separately based on AASHTO T 19. These unit weights are used to understand the volume of solids and voids in each fraction of each stockpile. Typical void ranges for LUW and RUW are 43 to 49 percent and 37 to 43 percent voids in the aggregate, respectively, for coarse aggregate, and 35 to 43 percent and 28 to 36 percent voids in the aggregate for fine aggregates (Aurilio, et. al., 2005).

Once the LUW and RUW are determined they are used to pick a chosen unit weight, CUW, which determines the volume of coarse aggregate in the blend. The CUW parameter is selected by the designer and determines if the blend is coarse or fine-graded as shown in Figure 3, modified from Vavrik, et. al., (2002). The Bailey Method designates mixes as coarse or fine based on the CUW of the gradation which is different from the Superpave definition based on percent passing the primary control sieves, PCS. (The PCS terminology is common to the Superpave and Bailey methods, but different definitions are used).Aurilio, et. al. (2005) recommended that if designing for a fine graded mix, the CUW should be 90 percent of the LUW or less. Conversely, if designing a coarse-graded mixture, one should use a CUW ranging from 95 to 105 percent of the LUW. A third type of mixture that can be used is known as stone mastic asphalt, SMA, and if this type of mix is being designed, the recommended range is 110 to 125 percent of the RUW. In addition to these recommendations, Vavrik, et. al. (2002) states that the range of 90 to 95 percent of LUW should be avoided for all dense-graded mixtures due to the probability that they will move in and out of coarse aggregate interlock during compaction.

In a coarse-graded mix the load is predominately carried by the interlock of the coarse aggregate and the fine aggregate serves as filler in the voids created by the coarse aggregate, in addition to providing some degree of load carrying strength. In a fine-graded mix the load is predominately carried by the fine aggregate. This means that there is not enough coarse aggregate in the blend to develop a skeleton and instead the coarse aggregate are essentially floating within the fine aggregate structure (Vavrik, et. al., 2002).

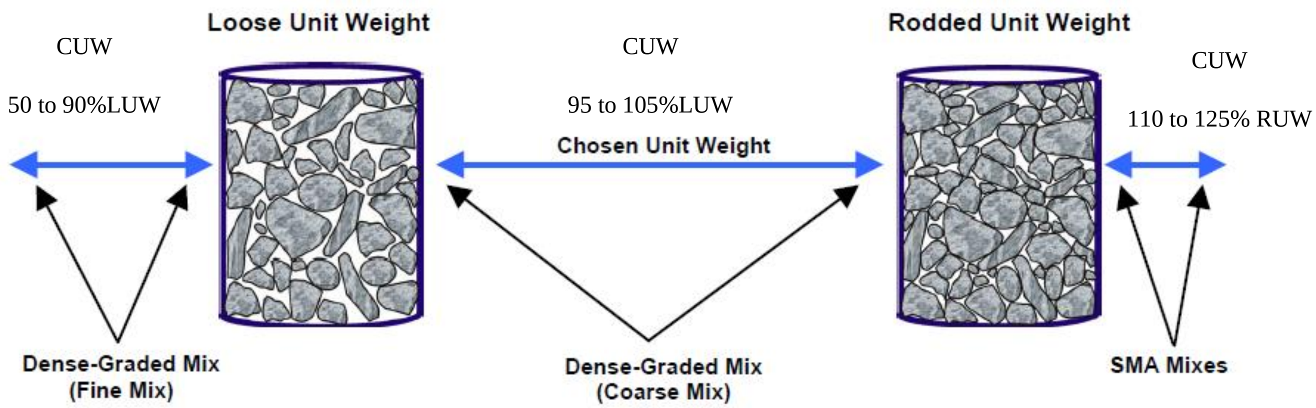

Figure 3: Selecting chosen unit weight for coarse aggregate

Once the unit weights have been determined and the CUW has been picked for the mix design the aggregate gradation can be analyzed. Analysis consisting of the four principles previously mentioned allows the designer to make necessary adjustments to the aggregate fractions to ensure strength and durability of the pavement based on the achieved aggregate skeleton and VMA.

2.7.2.1 Bailey Method Principle 1

Traditionally, the separation between coarse and fine aggregate is fixed at the 4.75 mm (No. 4) sieve, regardless of the NMAS of the aggregate blend. The Bailey Method defines the split between coarse and fine fractions of aggregate based on the particle packing principles. The Bailey Method uses the terminology nominal maximum particle size, NMPS, which is defined the same as the NMAS used for Superpave and therefore these two may be used interchangeably. In the case of a Bailey defined fine mix, however, a new NMPS is defined as the original PCS of the blend. The NMPS of the blend is multiplied by the particle size ratio of 0.22 and the closest sieve size to that value is designated as the primary control sieve, PCS, as shown in Table 7

(Aurilio, et. al., 2005).Aggregates passing the PCS are designated as the fine fraction and aggregates retained on the PCS make up the coarse fraction.

Table 7: Bailey primary control sieves

Mixture NMAS	NMAS X 0.22	Primary Control Sieve
37.5 mm	8.250 mm	9.5 mm
25.0 mm	5.500 mm	4.75 mm
19.0 mm	4.180 mm	4.75 mm
12.5 mm	2.750 mm	2.36 mm
9.5 mm	2.090 mm	2.36 mm
4.75 mm	1.045 mm	1.18 mm

2.7.2.2 Bailey Method Principle 2

The second principle of the Bailey Method refers to the coarse aggregate ratio or CA Ratio. The CA Ratio utilizes a “half sieve” which is defined differently depending on whether the mix is fine or coarse-graded. If dealing with a fine graded mixture, the half sieve is defined as the sieve closest to half of the PCS. The original PCS would then be considered the NMPS and a New PCS is calculated as 0.22 times the original PCS. For coarse-graded mixtures, the half sieve is half of the NMPS. Table 8 shows the half sieve that corresponds to the NMPS of a mix. Standard sieves that these are based on can be found in Table 1.

Table 8: Half sieves

NMPS	0.5 X NMPS	Half Sieve
37.5	18.75	19.0
19.0	9.5	9.5
12.5	6.25	4.75
9.5	4.75	4.75
4.75	2.375	2.36

Aggregates retained on the half sieve are referred to as pluggers, while aggregate passing the half sieve but retained on the PCS are termed interceptors (Aurilio, et. al., 2005). Interceptors are larger than the voids created by the pluggers and therefore limit the ability of the larger particles to achieve particle to particle contact. The CA Ratio is calculated as:

$$\boldsymbol{CA\ Ratio = \frac{\%\ passing\ half\ sieve - \%\ passing\ PCS}{100 - \%\ passing\ half\ sieve}} \qquad \text{Equation 3}$$

Vavrik, et. al. (2002) explains that, in general, as the CA Ratio increases, VMA will increase as well. However, as the CA Ratio approaches 1.0 the CA fraction will become unbalanced and the interceptors will begin to dominate the CA skeleton. This can cause compatibility problems in the field and therefore should be avoided. Table 9 outlines the concepts behind this principle and the effects on VMA with changes to the CA Ratio (Aurilio, et. al., 2005).

Table 9: Expected characteristics of blend based on CA Ratio

Fine Mix	Coarse Mix
Half sieve = half original PCS. (Original PCS becomes NMPS)	Half sieve = half NMPS
New PCS = 0.22 X original PCS (New CA Ratio calculated)	PCS remains unchanged
New coarse fraction is smaller than that of coarse mixtures and therefore less sensitive to changes	Coarse fraction larger than that of fine mixtures and more sensitive to changes
Too low of a new CA Ratio means there are not enough interceptors which results in low VMA and air voids	Too low of a CA Ratio means there are not enough interceptors which results in low VMA and air voids
Original CA Ratio of fine mixtures is not related to segregation	Too low of a CA Ratio means there are too many coarse particles and the mixture is prone to segregation
Too high new CA Ratio means there are too many interceptors and the mixture may be tender	Too high CA Ratio means there are too many interceptors and the mixture may be tender
New CA Ratio suggested range is 0.6 – 1.0	CA Ratio range depends of NMPS as recommended in Table 10(Vavrik, et. al., 2002)
0.35 increase in new CA Ratio creates approximately 1% increase in VMA and air voids	0.2 increase in CA Ratio creates approximately 1% increase in VMA and air voids

Table 10: Recommended CA Ratio ranges for coarse-graded mixtures

NMPS	37.5	25.0	19.0	12.5	9.5	4.75
CA Ratio	0.80-0.90	0.70-0.85	0.60-0.75	0.50-0.65	0.40-0.55	0.30-0.45

2.7.2.3 Bailey Method Principle 3

The third Bailey Method principle evaluates the effects of the coarse portion of the fine fraction of the blend. The fine fraction of the blend gets further broken down based on the concept of a secondary control sieve, SCS, in the same fashion as determination of the PCS. The PCS is multiplied by the particle packing ratio of 0.22 to determine the SCS. Aggregate retained on this SCS but passing the PCS is designated as the coarse portion of the fine fraction, and that which passes the SCS is the fine portion of the fine fraction. Similarly to the coarse fraction of a total blend, the coarse portion of the fine fraction creates voids that need to be filled with the fine portion of the fine fraction. Therefore, a fine aggregate coarse ratio, FA_c, is determined as (Vavrik, et. al., 2002):

$$\boldsymbol{FA_c} = \frac{\%\ \boldsymbol{passing\ SCS}}{\%\ \boldsymbol{passing\ PCS}} \qquad \text{Equation 4}$$

The effects of changes to the FA_c Ratio on VMA and compactability of a mixture are summarized in Table 11 (Aurilio, et. al. 2005).

Table 11: Expected characteristics of blend based on FA_c Ratio

Fine Mix	**Coarse Mix**
New SCS = 0.22 X new PCS	SCS = 0.22 X original PCS
New FA_c Ratio suggested range is 0.35 – 0.50	FA_c Ratio suggested range is 0.35 – 0.50
VMA begins to increase as new FA_c Ratio exceeds 0.50	VMA begins to increase as FA_c Ratio exceeds 0.55
As new FA_c Ratio increases toward 0.50 compactability of fine fraction increases	As FA_c Ratio increases toward 0.55 compactability of fine fraction increases
0.05 increase in new FA_c Ratio up to 0.50 results in approximately 1% decrease in VMA or air voids	0.05 increase in FA_c Ratio up to 0.55 results in approximately 1% decrease in VMA or air voids

2.7.2.4 Bailey Method Principle 4

The fourth Bailey principle evaluates the fine aggregate fine portion of the blend, FA_f. This principle uses a third or tertiary control sieve, TCS, to further break down the fine portion of the fine fraction. Similar to the FA_c Ratio, the FA_f Ratio is determined as (Vavrik, et. al., 2002):

$$\boldsymbol{FA_f} = \frac{\%\ \boldsymbol{passing\ TCS}}{\%\ \boldsymbol{passing\ SCS}} \qquad \text{Equation 5}$$

The FA_f Ratio evaluates the packing of the fine portion of the mixture. A summary of the characteristics of the mix with changes to the FA_f Ratio is shown in Table 12 (Aurilio, et. al., 2005). Once all of these ratios have been determined and evaluated the designer can use them to calculate adjustments to the trial gradation when making changes to the mix making the process less of a trial and error procedure and a more analytical method.

Table 12: Expected characteristics of blend based on FA_f Ratio

Fine Mix	**Coarse Mix**
New TCS = 0.22 X new SCS	TCS = 0.22 X original SCS
New FA_f Ratio suggested range is 0.35 – 0.50	FA_f Ratio suggested range is 0.35 – 0.50
VMA decreases as the new FA_f Ratio increases to 0.50	VMA decreases as the FA_f Ratio increases to 0.55
VMA begins to increase as new FA_c Ratio exceeds 0.50	VMA begins to increase as FA_c Ratio exceeds 0.55

Zaniewski and Mason (2006) used these four principles to evaluate the ability of the Bailey Method to predict the VMA. An Excel spreadsheet was created to aid with the use of the Bailey Method which automatically calculates blend percentages, aggregate ratios and gradation, as well as, provides guidance in selecting the CUW and whether or not a chosen gradation meets the aggregate ratio recommendations. Figure 4 shows a screenshot of the Bailey Method Excel spreadsheet user interface (Zaniewski and Mason, 2006).

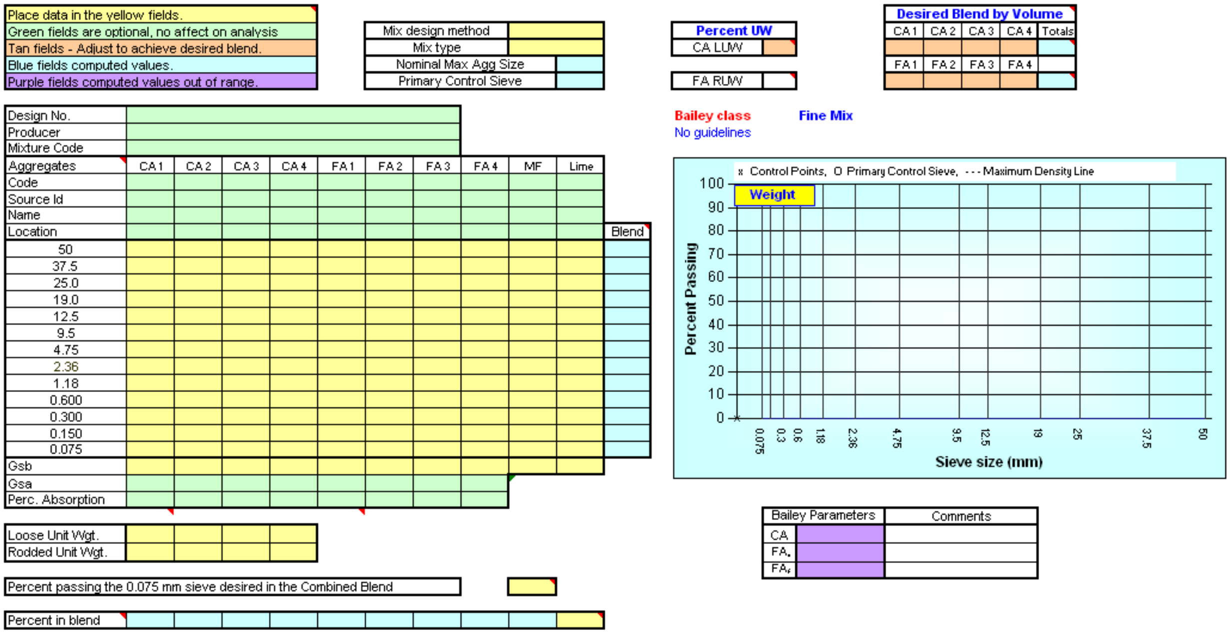

Figure 4: User interface for Bailey Method Excel spreadsheet

2.8 Locking Point Concept

The “locking point” concept was originally proposed by William Pine while working at the Illinois Department of Transportation. It is a concept that attempts to define a change in the compaction characteristics of asphalt concrete when being compacted in the SGC. The locking point is the point where the asphalt concrete mixture begins to develop an aggregate skeleton that resists further compaction. Original definition of the locking point for asphalt concrete is the first instance of three consecutive gyrations with no change in height, immediately preceded by two sets of two gyrations with the same height (2-2-3) (Vavrik, 2000). There are alternate definitions of the locking point including: the first instance of two consecutive gyrations with no change in height (2-), two sets of two consecutive gyrations at the same height (2-2), and three sets of two consecutive gyrations at the same height(2-2-2) (Li and Gibson, 2011). Prowell and Brown (2007) evaluated all four locking point definitions to compare the calculated locking point density versus 2-year old in place densities. Their findings indicated that the original definition (2-2-3) had the best relationship with ultimate. They also determined that the original definition appears to be the most conservative, meaning that it requires the most compaction effort to achieve.

Mohammad and Shamsi (2007) analyzed asphalt concrete mixtures with aggregate gradations designed with the Bailey Method. Limestone, sandstone and granite aggregates were evaluated for 12.5 mm NMAS mixes. Locking points were determined for each mixture and then compared against the Superpave recommended N_{des} levels. Additionally, the asphalt concrete mixtures were tested to determine their laboratory performance properties. Results of the locking point analysis determined that current Superpave recommended N_{des} levels, Table 4, were much higher than the locking points and could in turn be subjecting the asphalt concrete to unnecessarily high levels of compaction energy. This level of compaction energy can cause degradation of the aggregate and result in premature failure of the asphalt concrete. They concluded that mixtures with dense graded aggregate structures can be designed with their locking point instead of the N_{des} level recommended by Superpave. These mixtures demonstrate adequate durability and resistance to permanent deformation as measured by the Hamburg wheel tracking test and semicircular fracture and IT strength tests (Mohammad and Shamsi, 2007).

Li and Gibson (2011) evaluated multiple definitions of the locking point and attempted to relate them to mechanical performance tests. The three locking points they evaluated included: the first occurrence of three gyrations at the same height (3-), the second occurrence of two consecutive gyrations at the same height (2-2) and the third occurrence of two consecutive gyrations at the same height (2-2-2). Specimens were compacted to the various locking point definitions and evaluated for flow number, axial, radial and volumetric strains and solvent extraction to determine aggregate degradation. Their results showed that the locking point definition that produced the best results in terms of stability, strength and minimizing aggregate degradation was the 2-2 method (Li and Gibson, 2011). This method always results in a lower gyration level than the 2-2-3 method proposed by William Pine which would indicate that the 2-2-3 method may result in aggregate degradation in the SGC.

Vavrik (2000) studied the effects of changes in each fraction of the gradation as defined by the Bailey Method on the 2-2-3 locking point of a mixture. He indicates that for gradations with a low CA Ratio, the locking point is very sensitive to changes in the CUW of the blend. He also found that changes to the fine aggregate gradation affect the locking point of the mixture. Both the Bailey Method ratios and the locking point are indicators of an aggregate blend's packing characteristics. Therefore, Vavrik made a correlation between the two and developed an equation to predict the locking point of a mixture based on the Bailey Method ratios. Through experimentation he compared his predicted locking point values with measured valued and the results were accurate at predicting the locking point of a mixture with an R-squared value of 0.81 (Vavrik, 2000).

$$\boldsymbol{LP = 50.1 + 526.3CA^2 - 240.8CA + 1579.2\,FA_c^2 - 1283.7FA_c - 1734.3FA_f^2 - 1543.0FA_f}$$

Equation 6

Utilizing the locking point when designing an asphalt concrete mixture allows the desired volumetrics to be reached at the point where the aggregate structure is formed without over compacting and degrading the aggregate in the mixture. Also, by utilizing the Bailey Method and the equation provided by Vavrik, a designer can theoretically make adjustments to the aggregate blend and calculate the effects on volumetrics and locking point, minimizing some of the trial and error process to achieve a desired mix.

2.9 Indirect Tensile Strength

Currently there is no required laboratory proof test to evaluate the rutting susceptibility of Superpave asphalt concrete mixtures. There have been various suggestions for testing the rutting susceptibility of asphalt concrete mixtures that require equipment not readily available to the WVDOH. Zaniewski and Srinivasan (2003) evaluated using the IDT measured with the Marshall Stabilometer and compaction parameters to predict the rutting potential of an asphalt mixture. Their study determined that the rutting potential of a mixture can be predicted using the IDT measured with the Marshall Stabilometer and compaction slope, k, determined as:

$$\boldsymbol{k = \frac{\%G_{mm,Ndes} - \%G_{mm,Nini}}{\log(N_{des}) - \log(N_{ini})} * 100}$$ Equation 7

Where: $\%G_{mm,Ndes}$ = percent G_{mm} at design compaction level

$\%G_{mm,Nini}$ = percent G_{mm} at initial compaction level

N_{des} = design number of gyrations

N_{ini} = initial number of gyrations

$$\%\boldsymbol{G_{mm,Ndes}} = \%\boldsymbol{G_{mm,Ndes}} * \frac{\boldsymbol{h_{des}}}{\boldsymbol{h_{ini}}}$$ Equation 8

$$\%\boldsymbol{G_{mm,Ndes}} = \frac{\boldsymbol{G_{mb}}}{\boldsymbol{G_{mm}}}$$ Equation 9

Where: G_{mb} = bulk specific gravity of mixture

G_{mm} = theoretical maximum specific gravity of mixture

h_{des} = height of sample at design number of gyrations

h_{ini} = height of sample at initial number of gyrations

The IDT was determined using a Marshall Stabilometer load frame with and IDT load head per AASHTO T 283. The peak load was used to compute the tensile strength as:

$$IDT\ Strength = \frac{2P}{\pi t D} \qquad \text{Equation 10}$$

Where: P = maximum load (lbf)

t = specimen thickness (in)

D = specimen diameter (in)

As expected, they found a negative relationship between IDT and rutting potential determined with the Asphalt Pavement Analyzer, APA. As the IDT strength increases, the measured rut depth decreases. The results of rut depth versus IDT strength only are shown in Figure 5 along with the equation to predict rut depth with an R^2 value of 0.78.

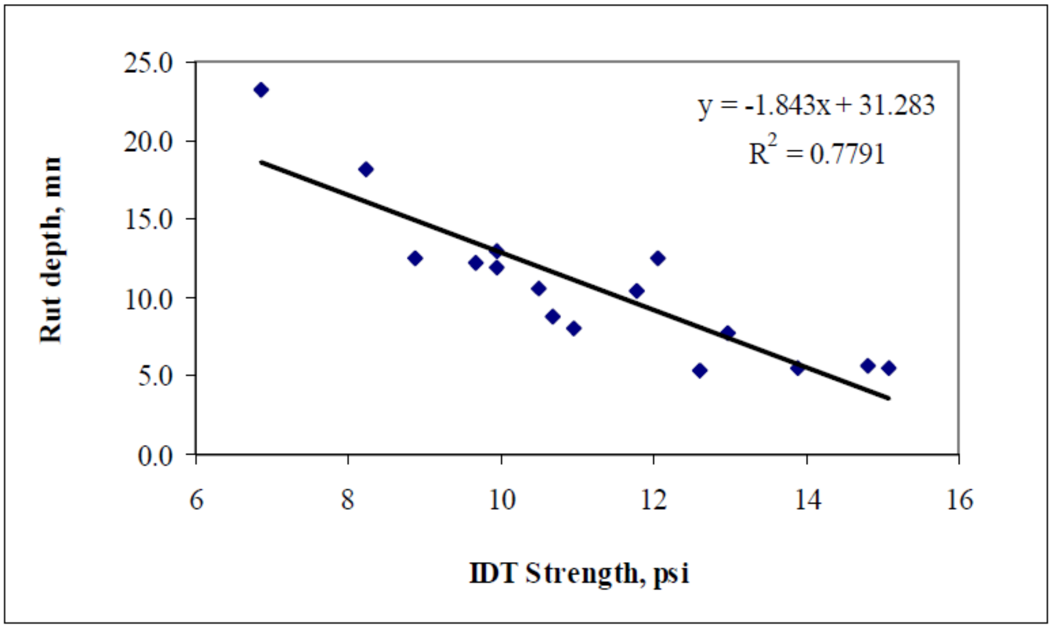

Figure 5: Rut depth versus IDT strength

Zaniewski and Srinivasan (2003) also analyzed the effects of covariant terms on the resulting rut depth of an asphalt concrete mixture. The best correlation to be IDT strength and k value on rutting potential with good predictive ability against measured values with R^2 value of 0.848. The regression equation was:

$$R_d = 10.36 + 2.45 \times k - 2.04 \times IDT \qquad \text{Equation 11}$$

Predicted rut depth versus APA measured rut depth (mm) is shown in Figure 6.

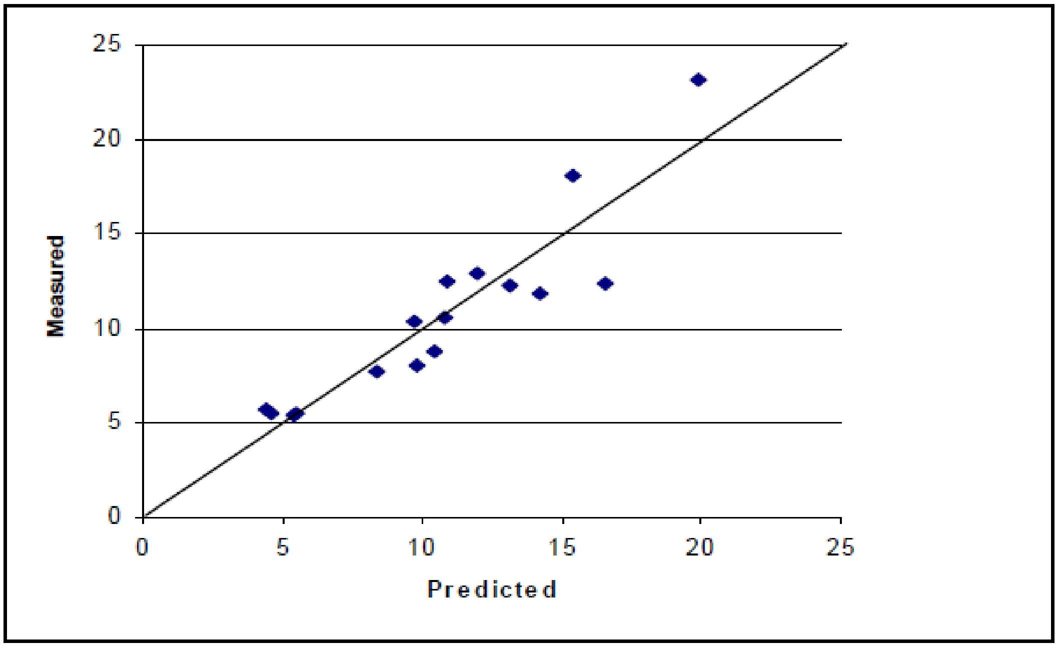

Figure 6: Predicted versus measured rut depth (mm)

2.10 Application of Literature to Thesis

For this thesis, the Bailey Method and ASTM D4253-00 were used to evaluate changes in aggregate gradation for each asphalt mixture. Locking points 2-2-3 and 2-2 were used for analysis because 2-2-3 was the original definition recommended by Vavrik (2000) and the 2-2 definition was determined by Li and Gibson (2007) to be the most representative of ultimate density of the pavement. Superpave volumetric calculations were used to draw inferences about the results of this experiment and whether or not it is a valuable tool in asphalt mix design. VMA of the mixes calculated with the Superpave equation was compared to the VMA determined with the dry density tests and compared to determine if the dry density test was an acceptable tool for predicting the VMA of a mix. IDT strength was also tested and evaluated for its relationship with other parameters tested in this thesis including CA ratio and CUW.

Chapter 3 RESEARCH METHODOLOGY

3.1 Introduction

The intent of this research was to evaluate the use of unit weights in choosing aggregate gradation for a mix design. Along with this goal, locking point, IDT strength and the Bailey Method were used to evaluate the mixtures. This experiment was organized to hold all factors that could affect the strength constant with exception of locking point and gradation in order to obtain desired confidence in the results. With this in consideration samples were made in accordance with the experimental design as described in the following sections.

3.2 Experimental Design

Aggregates for use in this experiment were obtained from Greer Limestone and Jefferson Asphalt. Each supplier provided current mix designs for a 9.5 mm and a 12.5 mm mix. The data needed for analysis, such as the bulk specific gravity and gradation of each stockpile, were taken from the contractors mix designs. From these mix designs, Bailey defined coarse and fine gradation mixes were developed for the experiment. The constraints used for selecting the stockpile blends were:

1. The blends satisfy the control limits for Superpave, but were shifted as far as possible away from the maximum density line, while meeting other constraints.
2. The blends were formulated by adjusting the percent of stockpiles in the contractors' mix design, i.e. the contractors could produce the coarse and fine blends from their existing stockpiles.

3. The Bailey Method was used for the final selection of the blends. Every effort was taken to satisfy the aggregate ratio ranges recommended by the Bailey Method; however, this was not always achievable with the given stockpiles.

This resulted in 12 mix designs and with three pills for IDT analysis and samples for dry density, a total of 72 test samples were required as shown in Table 13. To minimize variability all of the mix needed for a combination of contractor/mix type/gradation was mixed as a single batch. The batch was split to provide the material needed for each sample. The samples for volumetric analysis and IDT were compacted with 80 gyrations per the mix design. The samples for locking point analysis were compacted to 125 gyrations. To avoid bias based on order of experimentation the mix designs were each assigned a random number using an Excel function and samples were mixed, compacted and tested in the random order as shown in Table 14.

Table 13: Experimental design

			Gradation								
			Contractor			Coarse			Fine		
Greer Limestone	9.5	Dry ρ	1	2	3	7	8	9	13	14	15
		IDT	4	5	6	10	11	12	16	17	18
	12.5	Dry ρ	19	20	21	25	26	27	31	32	33
		IDT	22	23	24	28	29	30	34	35	36
Jefferson Asphalt	9.5	Dry ρ	37	38	39	43	44	45	49	50	51
		IDT	40	41	42	46	47	48	52	53	54
	12.5	Dry ρ	55	56	57	61	62	63	67	68	69
		IDT	58	59	60	64	65	66	70	71	72

Table 14: Random sample preparation and testing order

			Gradation		
			Contractor	Coarse	Fine
Greer Limestone	9.5	Dry ρ / IDT	7	10	3
	12.5	Dry ρ / IDT	12	11	4
Jefferson Asphalt	9.5	Dry ρ / IDT	2	6	1
	12.5	Dry ρ / IDT	9	8	5

3.3 Mix Designs

In preparation for mixing, all aggregate stockpiles were sieved, washed and dried to allow maximum control of aggregate gradation in the mix designs. Gradation curves for each mix design are in the Appendix. Mix designs for the coarse and fine gradations were then carried out by adjusting the contractors asphalt content by -0.2 percent when changing to a coarse mix and +0.3 percent when changing to a fine mix. If the mix did not meet the Superpave volumetric requirements, the Superpave equations for adjusting percent binder were used and new samples were made. Samples of the contractors mixes were made to ensure their volumetric properties could be reproduced in the WVU Asphalt Technology Laboratory. Table 15 and Table 16 summarize the mix design parameters for Greer and Jefferson respectively.

3.4 Evaluation of the Mixes

The evaluation of the mixes included estimating the aggregate density in the pills and measuring the IDT strength. One sample for each mix type was compacted to N_{max} for the locking point evaluation where:

$$N_{max} = {N_{des}}^{1.1} = 80^{1.1} = 125\ gyrations$$

Table 15: Greer mix design parameters

	Greer					
	9.5 mm			12.5 mm		
% Stockpile in Blend	Contractor (Bailey Fine)	Coarse	Fine	Contractor (Bailey Coarse)	Coarse	Fine
Buckeye #7				35	38	24
Buckeye #8	45	56	18	20	22	14
Buckeye Sand	40	34	60	30	23	42
Greer Sand	15	10	22	15	17	20
P_b	6.2	6.2	6.4	5.9	6.1	6.1
VTM	3.3	3.9	3.9	4.0	4.9	3.6
VMA	16.6	17.0	17.0	16.8	17.6	16.2
VFA	80	77	77	76	72	78
D/B	0.9	0.8	1.2	0.8	0.7	1.0
CA	0.72	0.55	2.07	1.09	0.95	1.57
FA_c	0.42	0.41	-	0.42	0.39	-
FA_f	0.5	0.54	-	0.54	0.55	-
CA_{new}	0.69	-	0.6	0.64	-	0.6
$FA_{c,new}$	0.5	-	0.45	0.54	-	0.47
$FA_{f,new}$	-	-	-	-	-	-

3.4.1 Aggregate Density of the Asphalt Samples

Once three acceptable pills based on the Superpave criteria were made for each mix design the aggregate density was determined. This was achieved by determining the volume of the pill using the diameter of the mold and the pill height provided by the SGC. Multiple pills were spot checked for actual height to confirm the height data provided by the compactor. Percent stone was then multiplied by the dry mass of the bulk pill to determine the mass of the aggregate in the pill and this was divided by the sample volume to determine the aggregate density in the mix.

Table 16: Jefferson mix design parameters

	Jefferson					
	9.5 mm			12.5 mm		
% Stockpile in Blend	Contractor (Bailey Fine)	Coarse	Fine	Contractor (Bailey Fine)	Coarse	Fine
MF Sand	25	21	59	30	16	43
Inwood #10	30	17	8			
Agg Industries #10				20	20	17
Diabase #8	23	10	17	25	21	11
Dolamite #7				25	43	29
Dolamite #8	22	52	16			
P_b	5.5	6.0	6.3	5.1	5.4	5.3
VTM	4.3	3.9	3.2	3.5	3.0	3.6
VMA	15.9	16.7	17.1	14.6	15.7	15.3
VFA	73	76	81	76	81	76
D/B	1.0	0.6	0.9	1.1	0.8	1.1
CA	0.54	0.4	0.5	0.65	0.55	0.64
FA_c	0.39	0.38	-	0.39	0.41	-
FA_f	0.39	0.38	-	0.41	0.46	-
CA_{new}	0.65	-	0.71	0.69	-	0.7
$FA_{c,new}$	0.39	-	0.36	0.41	-	0.4
$FA_{f,new}$	-	-	-	-	-	-

3.4.2 VMA of Samples

In addition to the Superpave defined VMA, an additional VMA was determined using the bulk volume of the pill which included the surface voids on the outside of the pill. The reason for this “bulk volume VMA” was to have a comparison to the dry VMA calculated from the dry density of the aggregate as explained in section 3.5. These surface voids are not included when calculating the VMA using the typical volumetric calculations and therefore it was decided that this additional volume VMA was needed for comparison.

3.4.3 IDT Strength Testing

The indirect tensile strength of each pill was determined using the Marshall loading frame with a modified head. Each pill was heated in a water bath at 60°C for one hour and fifteen minutes to achieve a constant temperature. The pills were then placed in the Marshall Stabilometer and loaded at a rate of 50 mm per minute. The IDT strength was computed from the maximum load and sample dimensions per Equation 10.This information was used to evaluate the mixes for strength and compare that to the results of the mix meeting the Bailey ratio criteria or not. The goal of this was to determine whether or not a mix performs better in terms of IDT strength if it meets the ratio criteria specified in the Bailey Method.

3.4.4 Locking Point

For each mix design, one pill was compacted up to N_{max} of 125 gyrations to determine the locking point for the mix. Multiple definitions of locking point were evaluated including:

- the first gyration of three gyrations at the same height immediately preceded by two sets of two gyrations at the same height (2-2-3)
- two gyrations at the same height (2-)
- two gyrations at the same height followed by two more at the same height (2-2)
- three instances of two gyrations at the same height (2-2-2)
- the first instance of three gyrations at the same height (3-)

Figure 7 shows the various locking points on a SGC height data curve.

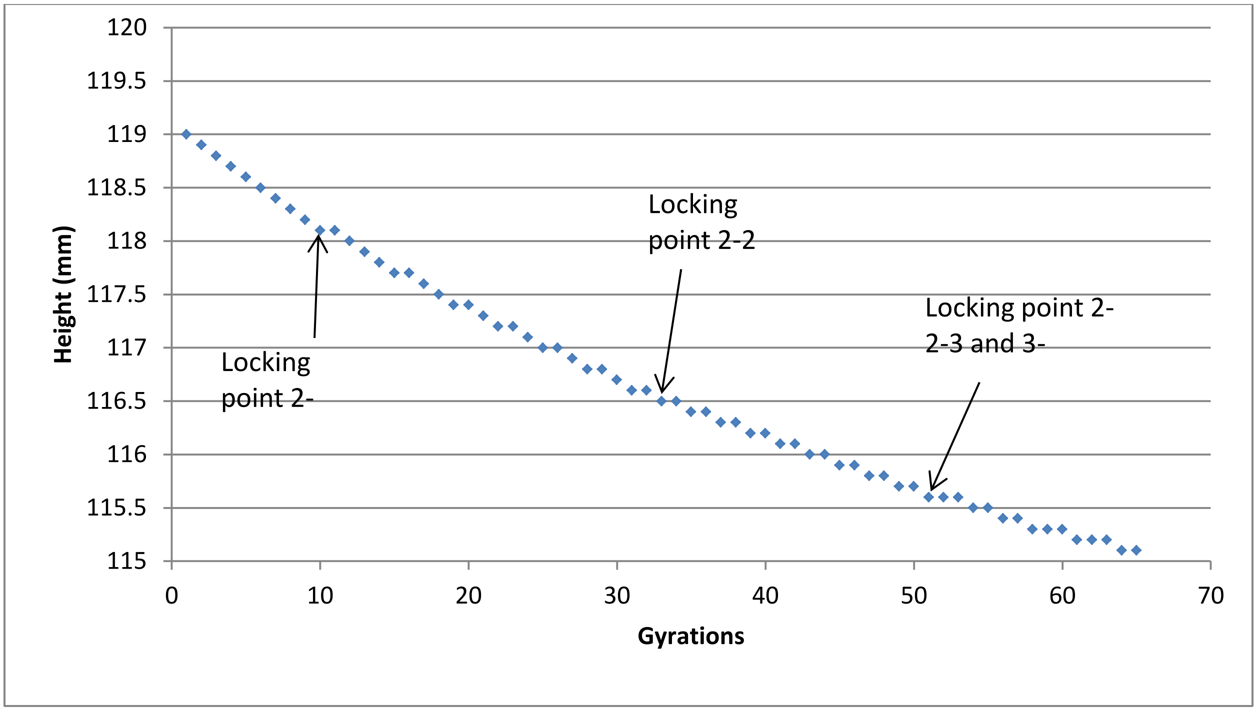

Figure 7: Diagram showing locking points on a SGC compaction curve

3.5 Dry Density Analysis

Each gradation that was chosen using the Bailey Method was weighed out and mixed to achieve a uniform blending of the aggregates. The blend was then placed into the mold as described in ASTM D4253-00. A surcharge base plate, guide sleeve and surcharge weight were placed on the sample to and the entire apparatus was placed on a vibrating table as specified in ASTM D4253-00. The sample was then vibrated for 8.25±1 minutes to ensure maximum compaction of the aggregate sample. Volume was determined by measuring from the top of the mold to the surcharge base plate in four locations on opposite sides of the mold. The thickness of the plate was added to these measurements and the four heights were averaged. This value was subtracted from the height of the mold and the volume of the sample was determined. The mass of the aggregate in the sample was calculated by subtracting the mass of the empty mold from the mass of the vibrated sample and mold. Maximum density was determined by dividing the mass of the aggregate by the volume that the aggregate occupied after vibration to determine "dry density." Additionally, VMA of the vibrated sample was calculated by determining the stone volume using the bulk specific gravity of the stone and subtracting that from the sample volume. This was divided by the total sample volume to determine "dry VMA."

3.6 Effective Binder Volume

The Superpave effective binder volume, V_{be}, equation is defined as:

$$\boldsymbol{V_{be} = 0.176 - 0.0675\log(NMAS)} \quad \text{Equation 12}$$

Since the only variable in this equation is the nominal maximum aggregate size, NMAS, the predicted V_{be} will be the same for any mix with the same NMAS regardless of the blend gradation. VMA is the sum of the percent of effective binder plus the air voids. This leads to the rationale that as the VMA changes, the V_{be} changes as well. Therefore, the V_{be} predicted from the Superpave equation, as well as, a predicted V_{be} from the dry density test was compared to the actual V_{be} in the pills to evaluate the accuracy of these two methods.

Chapter 4 RESULTS AND ANALYSIS

4.1 Introduction

The purpose of this chapter is to present the results obtained by the laboratory testing as presented in Chapter 3, and subsequently analyze these results to draw conclusions on the various topics being studied, including; aggregate density, IDT strength and locking point. Based on the results obtained from the dry density tests and the laboratory pills, inferences were drawn on the interactions of various mix parameters and supported with statistical analysis. Additionally, a comprehensive discussion of the meaning of the results is also provided.

4.2 Test Results

The results are presented in Table 17, these values are averages for each mix type of the individual samples made during the testing procedure. Results for each sample are found in the Appendix. Each topic being analyzed is discussed in the following sections and is based in the data shown in Table 17.

4.3 Analysis

The subjects evaluated in this thesis include: aggregate density, locking point, voids in mineral aggregate, VMA, the Bailey Method and the Superpave estimates for initial percent binder in an asphalt mixture.

Table 17: Data summary

	Greer						Jefferson					
	9.5 mm			12.5 mm			9.5 mm			12.5 mm		
	Contractor (Bailey Fine)	Coarse	Fine	Contractor (Bailey Coarse)	Coarse	Fine	Contractor (Bailey Fine)	Coarse	Fine	Contractor (Bailey Fine)	Coarse	Fine
P_b	6.2	6.2	6.4	5.9	6.1	6.1	5.5	6.0	6.3	5.1	5.4	5.3
VTM	3.3	3.9	3.9	4.0	4.9	3.6	4.3	3.9	3.2	3.5	3.0	3.6
Pill VMA	16.6	17.0	17.0	16.8	17.6	16.2	15.9	16.7	17.1	14.6	15.7	15.3
Pill Bulk VMA	21.0	21.6	21.0	21.4	22.4	20.5	20.3	21.5	21.0	19.1	20.8	19.5
Pill Volume (m^3)	0.00204	0.00206	0.00206	0.00208	0.00210	0.00206	0.00197	0.00201	0.00198	0.00198	0.00201	0.00201
Aggregate in Pill Density (kg/m^3)	2170	2158	2156	2162	2138	2180	2284	2269	2294	2368	2325	2346
IDT Strength (kN/m^2)	169.6	149.2	207.9	152.6	143.2	184.5	221.0	131.7	196.0	207.7	149.3	216.9
Dry VMA	21.1	21.3	21.2	22.1	22.4	20.6	18.8	20.7	18.1	19.1	21.6	17.6
Dry Density (kg/m^3)	2098	2098	2082	2077	2071	2108	2255	2222	2302	2293	2229	2327
Percent Max Dry Density (kg/m^3)	103.5	102.9	103.6	104.1	103.3	103.4	101.3	102.1	99.7	103.3	104.3	100.8
Locking Point 2-2-3	99	98	97	103	101	103	85	95	81	78	96	78
Locking Point 2-	62	61	63	61	60	62	57	60	52	48	55	50
Locking Point 2-2	79	81	82	82	82	85	72	76	68	69	73	64
Locking Point 2-2-2	91	88	89	89	89	87	74	78	70	71	80	71
Locking Point 3-	99	98	97	103	101	103	85	95	81	78	96	78
G_{mm}	2.447	2.455	2.439	2.456	2.462	2.458	2.581	2.583	2.570	2.645	2.614	2.619
G_{mb}	2.365	2.359	2.343	2.357	2.342	2.370	2.470	2.481	2.488	2.551	2.536	2.524
G_{sb}	2.660	2.667	2.643	2.665	2.668	2.655	2.775	2.801	2.812	2.835	2.845	2.823
G_{sa}	2.728	2.725	2.733	2.726	2.725	2.729	2.854	2.861	2.881	2.911	2.913	2.897
G_{se}	2.714	2.713	2.715	2.714	2.714	2.714	2.838	2.849	2.867	2.896	2.899	2.882
$V_{be, pills, percent}$	13.3	13.1	13.1	12.7	12.7	12.6	11.6	12.8	13.9	11.1	12.7	11.7
$V_{be, predicted, Superpave, percent}$	11.0	11.0	11.0	10.2	10.2	10.2	11.0	11.0	11.0	10.2	10.2	10.2

4.3.1 Aggregate Density

Aggregate density in the asphalt mixtures was analyzed by evaluating the maximum dry density of the aggregate blend based on ASTM D4253-00 versus the density of the aggregate in the pills. Figure 8 shows a line of equality plot of this data. A t-test comparing the two data sets indicated with a two tailed P-value of 0.144 that the null hypothesis of equal means cannot be rejected. However, a linear regression of the data shown on the graph indicated that the hypothesis of the slope being equal to 1 and the y-intercept equal to 0, indicating equal values, can both be rejected with p-values of 0.033 and 0.018 respectively.

One of the concerns that this thesis attempted to analyze was evaluating if the current 80 gyration asphalt mixtures were actually reaching a dense aggregate configuration within the mix. The concern was that when the specification was changed to reduce the design number of gyrations from 100 to 80 that the solution to achieving target 4% air voids was to simply add more asphalt. This could in effect prevent the aggregate from forming a strong interlocking aggregate structure and allow premature failure of the asphalt pavement. The results of this test indicate that the asphalt mixture does in fact have a dense aggregate configuration and is forming a strong aggregate structure.

Beyond the fact that the aggregate has formed a strong aggregate skeleton, it is clear that the aggregate in the compacted asphalt specimen is actually reaching a denser configuration than was achieved by the maximum dry density test. It was speculated that the hot asphalt actually acts as a lubricant when the sample is being compacted. The shearing action produced by the gyratory compactor causes the aggregate to slide past each other to achieve a dense configuration.

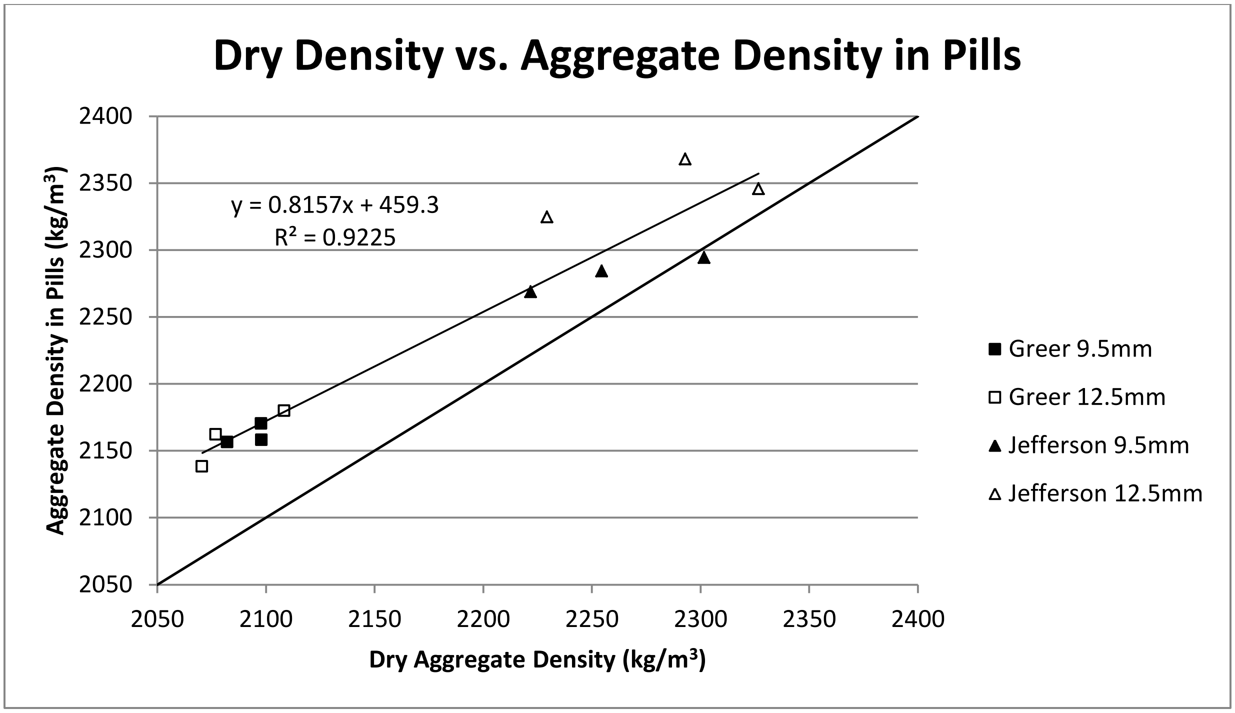

Figure 8: Dry density versus aggregate density in pills

4.3.2 Locking Point

The locking point of each mix was evaluated for comparison with the aggregate density and the IDT strength to evaluate the performance of the mix. Five different definitions of locking point were evaluated for these twelve mixes including the original definition of the first occurrence of three gyrations at the same height, immediately preceded by two sets of two gyrations at the same height. Based on the study by Li and Gibson (2011), the 2-2 locking point provided the best results based on mechanistic tests and aggregate degradation. If this were to be used for N_{des} the Greer mixes turned out very close to the locking point at the 80 gyration design level. However, for the Jefferson mixes to use the 2-2 locking point for N_{des} the design gyrations would have to be reduced to about 70.

The Jefferson mixes had lower locking points for all cases and in three out of six cases the locking point was lower than the design gyration level. Figure 9 plots the locking point against the aggregate density in the pills. There is a negative relationship with a R^2 value of 0.715 indicating that the locking point increases with a decrease in aggregate density in the mix. With the obvious separation of the aggregate suppliers in the graph, however, there is potential that there is a difference in other potentially significant variables such as aggregate angularity that could affect the locking point of the mixture.

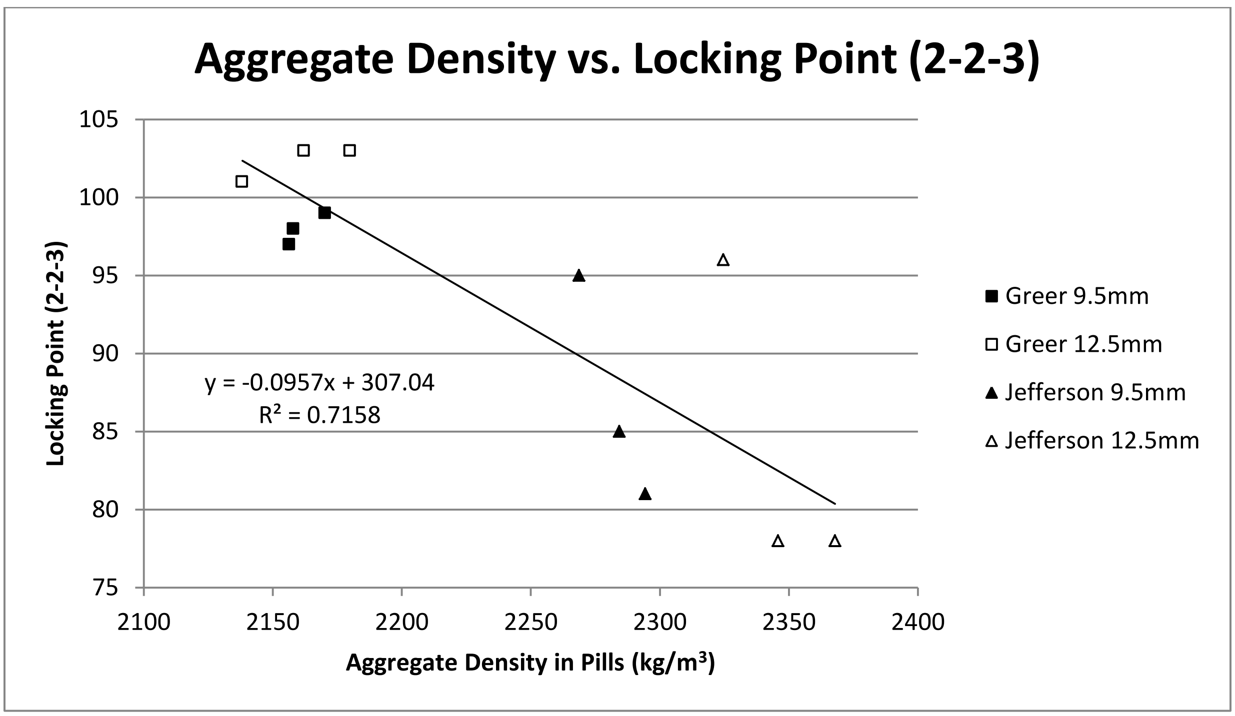

Figure 9: Aggregate density in the asphalt mixture versus locking point

The density of aggregate in the pills was evaluated when compacted to:

- locking point 2-2
- N_{des} (80 gyrations)
- locking point 2-2-3
- N_{max} (125 gyrations)

As shown in Figure 10 there is a positive trend in aggregate density with increasing gyrations. This is expected as the volume of the pill is decreasing while the mass of aggregate remains the same. This increase in density demonstrates that the aggregate is in fact becoming denser with increasing gyrations. However, based on the research by Li and Gibson (2011), beyond the 2-2 locking point the aggregate can degrade, meaning that there is exposed aggregate surfaces within the mixture that is not covered with asphalt. As show in Table 18, the Jefferson mixes had 2-2 locking points less than the N_{des} of 80 gyrations, while the Greer mixes had 2-2 locking points very close to the N_{des} gyration level. This indicates that the Jefferson mixes could potentially experience better performance if the N_{des} level was decreased to the 2-2 locking point.

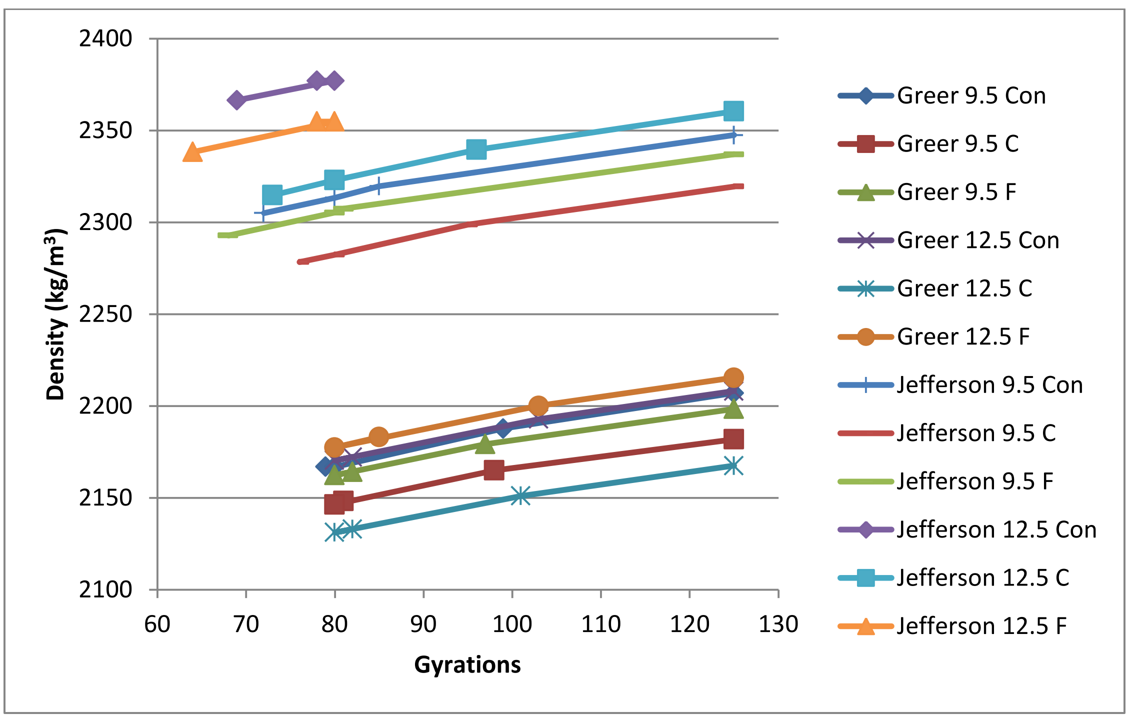

Figure 10: Aggregate density at varying gyration levels

Table 18: Aggregate density and locking point

	Dry Density **	Density at LP2-2	Density at N_{des}	Density at LP 2-2-3	Density at N_{max}	Locking Point 2-2	N_{des}	Locking Point 2-2-3	N_{max}
Greer 9.5 Con*	2098	2167	2167	2188	2207	79	80	99	125
Greer 9.5 C	2098	2148	2146	2165	2182	81	80	98	125
Greer 9.5 F	2082	2164	2162	2179	2198	82	80	97	125
Greer 12.5 Con	2077	2172	2170	2193	2208	82	80	103	125
Greer 12.5 C	2071	2133	2131	2151	2168	82	80	101	125
Greer 12.5 F	2108	2183	2177	2200	2215	85	80	103	125
Jefferson 9.5 Con	2255	2305	2313	2320	2347	72	80	85	125
Jefferson 9.5 C	2222	2278	2282	2299	2320	76	80	95	125
Jefferson 9.5 F	2302	2293	2305	2307	2337	68	80	81	125
Jefferson 12.5 Con	2293	2366	2377	2377	-	69	80	78	125
Jefferson 12.5 C	2229	2315	2323	2339	2360	73	80	96	125
Jefferson 12.5 F	2327	2338	2355	2355	-	64	80	78	125

* Con = contractors' blends, C = coarse blends, F = fine blends

** All densities in kg/m^3

The aggregate density in the pills at locking points 2-2-3, 2-2 and N_{des} were compared to the aggregate dry density, Figure 11. A t-test on the linear regression equation coefficients rejected the null hypothesis meaning that the coefficients are not equal to 1 and 0 for the slope and y-intercept respectively based on statistical analysis indicating that the aggregate density in the pills is not equal to the dry density. Figure 11 shows that the density of the aggregate at the 2-2 locking point is slightly closer to the line of equality that either the 2-2-3 locking point or N_{des}. This supports the recommendations by Li and Gibson (2011) that the 2-2 locking point is a better indicator of aggregate structure being formed. It could indicate that the Greer mixes are compacting to the correct level of 80 gyrations while the Jefferson mixes may be experiencing aggregate degradation by imposing excessive compaction force. A reduction to around 70 gyrations for the Jefferson mixes would be more comparable to the 2-2 locking point.

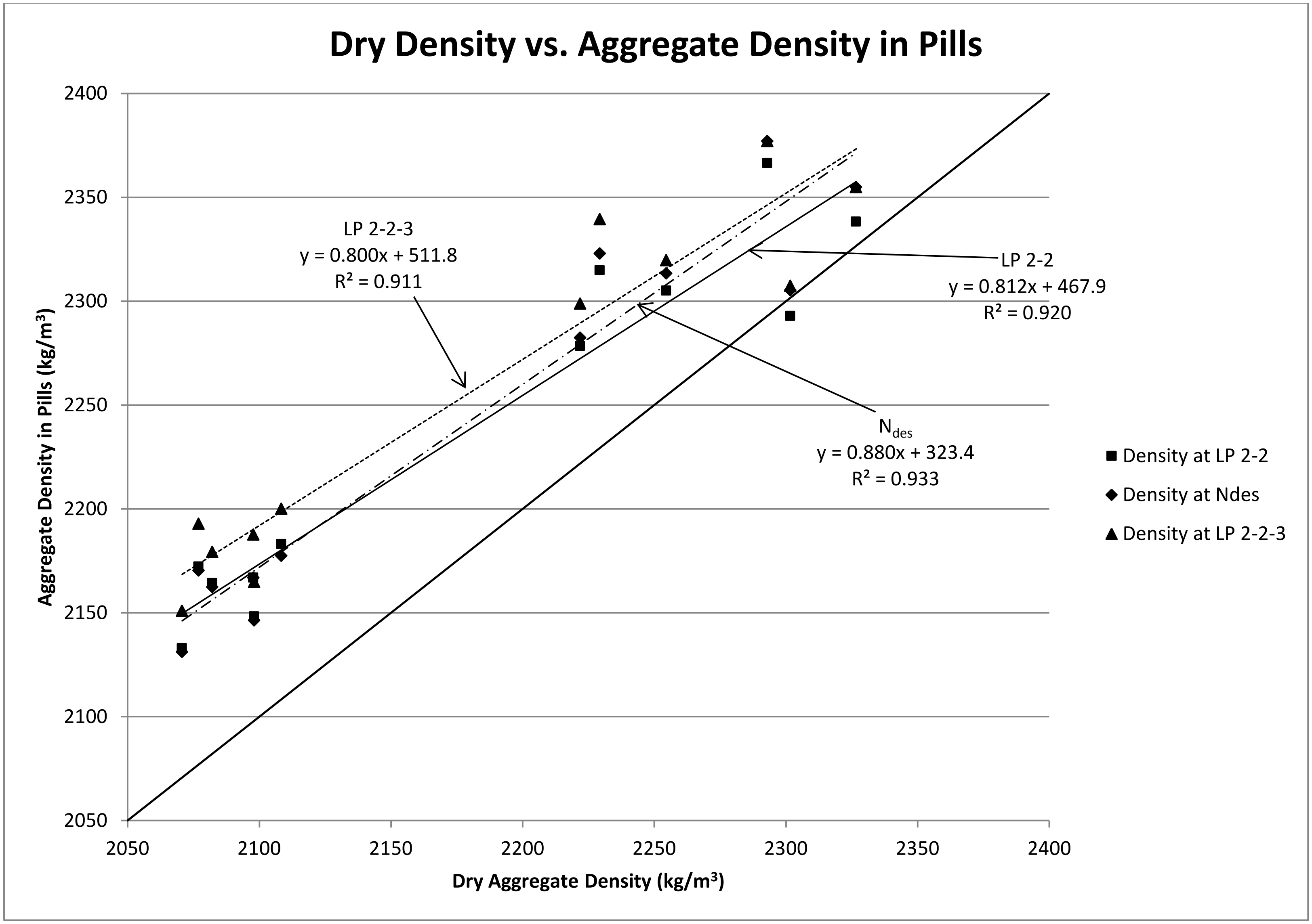

Figure 11: Dry density of aggregate versus aggregate density in pills at 2-2-3, 2-2, N_{des}

Equation 6 provided by Vavrik (2000) for calculating the locking point of a mix based on the Bailey ratios was evaluated. The values for locking point given by Equation 6 for the mixes in this thesis were incorrect as they all returned negative values. When the equation was checked with the aggregate ratio values from Vavrik's (2000) thesis, they also resulted in negative values for locking point. Equation 6 was altered by making the last term of the equation positive instead of negative and the values computed from Vavrik's (2000) thesis appeared correct. This adjusted equation was used for the mixes in this thesis, however, the values predicted for locking point were far from the actual locking points measured from the SGC data as shown in Table 19. Therefore, it was determined that Equation 6 provided by Vavrik (2000) was not an accurate predictor of the locking point of a mixture.

Table 19: Calculated locking point versus actual locking point

	CA	FA_c	FA_f	CA_{new}	$FA_{c,new}$	$FA_{f,new}$	LP_{Vavrik}	$LP_{Vavrik,adjusted}$	Locking Point 2-2-3
Greer 9.5 Con (F)*	0.72	0.42	0.50	0.69	0.50		-113	-113	99
Greer 9.5 C	0.55	0.41	0.54				-1523	144	98
Greer 9.5 F	2.07			0.60	0.45		-163	-163	97
Greer 12.5 Con (C)	1.09	0.42	0.54	0.64	0.54		-1187	480	103
Greer 12.5 C	0.95	0.39	0.55				-1337	360	101
Greer 12.5 F	1.57			0.60	0.47		-159	-159	103
Jefferson 9.5 Con (F)	0.54	0.39	0.39	0.65	0.39		-145	-145	85
Jefferson 9.5 C	0.40	0.38	0.38				-1059	114	95
Jefferson 9.5 F	0.50			0.71	0.36		-113	-113	81
Jefferson 12.5 Con (F)	0.65	0.39	0.41	0.69	0.41		-126	-126	78
Jefferson 12.5 C	0.55	0.41	0.46				-1261	159	96
Jefferson 12.5 F	0.64			0.70	0.40		-121	-121	78

* Con = contractors' blends, C = coarse blends, F = fine blends

4.3.3 Voids in Mineral Aggregate

The VMA of the asphalt mixtures were compared to the corresponding VMA of the dry density samples as shown in Figure 12. Along with a low R^2 value of 0.275 it is obvious that the two values are not equal as the values are not along the line of equality. This observation was supported with a t-test. For an alpha value of 0.05, the two-tailed P value is much lower than 0.05 meaning that the hypothesis of equal means is rejected. The reason these two values are different is because the dry density VMA is calculated using the bulk volume of the sample. The standard VMA definition does not account for the voids of the surface of the pill like the dry density VMA calculation, therefore, an additional "volume VMA" was defined which uses the volume of the pill including the surface voids.

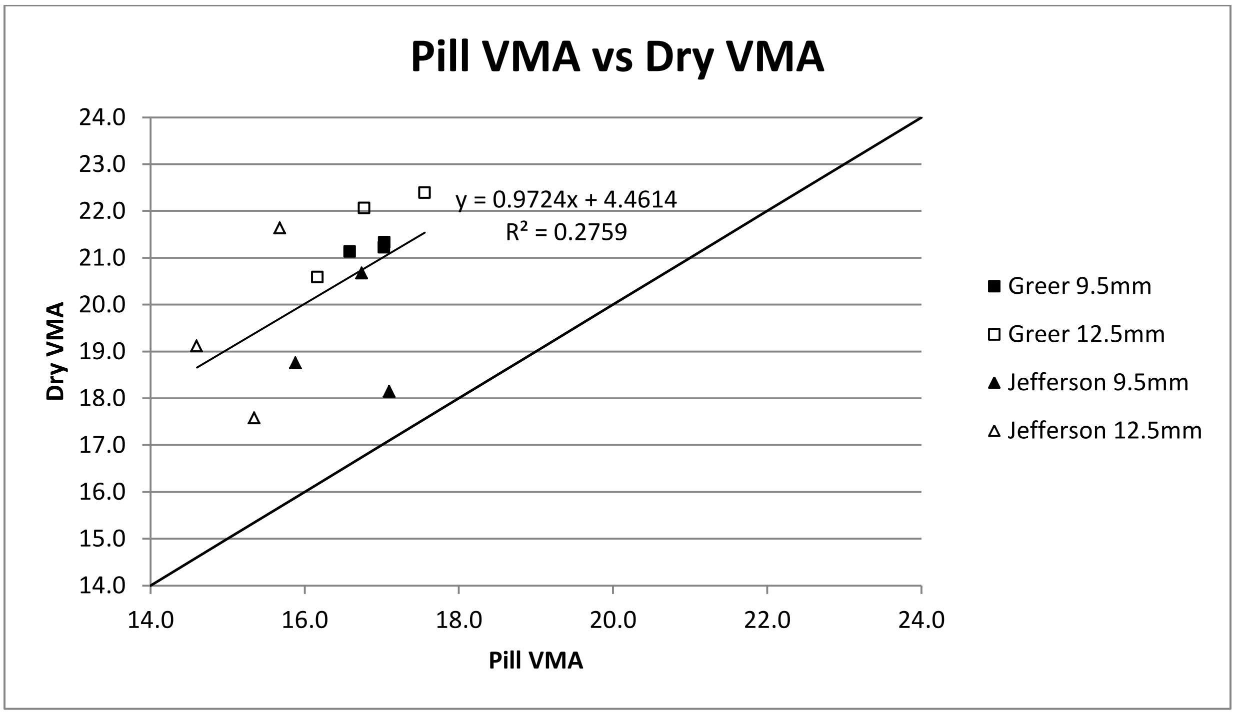

Figure 12: Pill VMA versus dry density VMA line of equality chart

The volume VMA was calculated as:

$$VMA_v = \frac{\pi h r^2 - {}^{M_s}/_{G_{sb}}}{\pi h r^2} \qquad \text{Equation 13}$$

This provided results much closer to the dry density VMA. A t-test returned a two-tailed P value of 0.41 which is much greater than 0.05, meaning that there is insufficient evidence to reject the hypothesis that the means of the two data sets are equal. Analysis of the coefficients in the linear regression equation in Figure 13 achieved the same conclusion. Inability to reject the null hypothesis is a result of the variability in the data. The R^2 value of 0.54 indicates that the variability in the data is not explained by the equation and therefore calling the two values the same would be inaccurate. An attempt was made to correlate the volume VMA to the dry density VMA in order to estimate the pill VMA based on the dry density. It was determined, however, that due to the variability, the VMA of the pill cannot be reliably estimated from the VMA of the dry density sample. Figure 13 shows a clear distinction for the results for the two contractors' materials. The Greer mix results are very close to the line of equality while the Jefferson mixes show a lot of scatter.

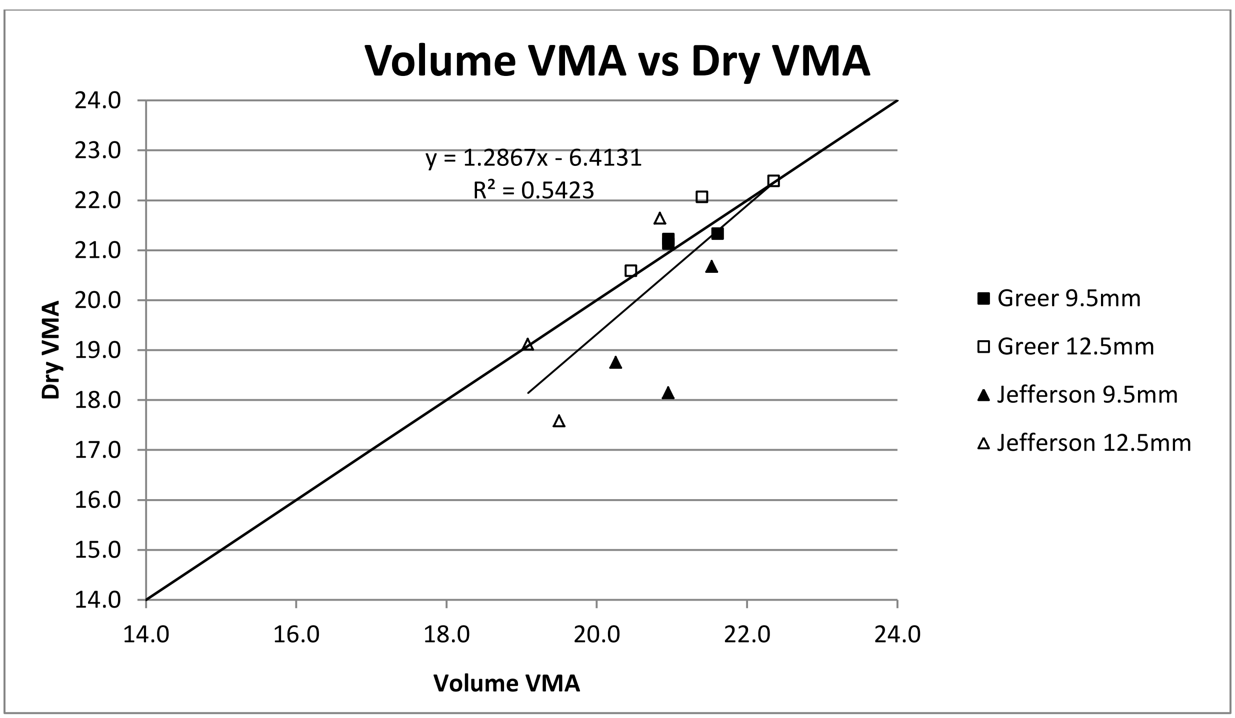

Figure 13: Volume VMA versus dry density VMA line of equality chart

The change in VMA from the contractors gradation based on the dry density test was compared to the actual change in VMA of the mixes. Table 20 shows the results of this comparison. The results of a t-test indicated there is insufficient evidence to reject the null hypothesis of equal means of the actual mix VMA and the dry VMA with a two-tailed P value of 0.44. However, it is clear from the data that this test does not always predict an accurate change in direction of the VMA as shown for the Jefferson 9.5 mm and 12.5 mm fine mixes. This leads to the recommendation that the use of the dry VMA cannot be effectively used as a prediction of VMA with a change in gradation.

Table 20: Change in VMA from dry density test versus actual mix VMA

Supplier	Mix Type	Gradation	Pill VMA	Dry VMA	Pill VMA Change	Dry VMA Change
Greer	9.5 mm	Contractor	16.6	21.1	-	-
Greer	9.5 mm	Coarse	17.0	21.3	0.4	0.2
Greer	9.5 mm	Fine	17.0	21.2	0.4	0.1
Greer	12.5 mm	Contractor	16.8	22.1	-	-
Greer	12.5 mm	Coarse	17.6	22.4	0.8	0.3
Greer	12.5 mm	Fine	16.2	20.6	-0.6	-1.5
Jefferson	9.5 mm	Contractor	15.9	18.8	-	-
Jefferson	9.5 mm	Coarse	16.7	20.7	0.9	1.9
Jefferson	9.5 mm	Fine	17.1	18.1	1.2	-0.6
Jefferson	12.5 mm	Contractor	14.6	19.1	-	-
Jefferson	12.5 mm	Coarse	15.7	21.6	1.1	2.5
Jefferson	12.5 mm	Fine	15.3	17.6	0.7	-1.5

4.3.4 Bailey Method

The Bailey Method was evaluated for its ability to predict VMA and performance of the mix. As discussed in Chapter 2, the Bailey Method provides guidelines for predicting the VMA based on changes in the aggregate ratios. The Bailey Method, however, does not provide a way to evaluate a mixture as it transitions from a fine mix to a coarse mix. When the contractor's Bailey defined fine mix was changed to a coarse mix, the changes in VMA were calculated assuming the coarse mix parameters. Alternately, if the contractor's coarse mix was being changed to a fine mix, the VMA changes were calculated assuming the fine mix parameters. This assumption is consistent with that used by Zaniewski and Mason (2006). Table 21 shows the Bailey predicted VMA compared to the actual VMA calculated for each mix. There is clearly a discrepancy in the prediction of VMA for the coarse mixtures. In every case, Bailey predicted a decrease in the VMA while the VMA of the pills increased. Additionally, for the Greer 12.5 mm mix, Bailey predicted that the VMA would increase while the VMA decreased. This predicted decrease in VMA is contrary to the notion that as the blend gradation curve moves further away from the maximum density line; the VMA would be expected to increase. This could be partially explained by the fact that the assumption was made that the coarse fraction was in control when changing from a contractor's fine mix, however, that explanation does not work for the Greer 12.5 mm coarse mix where the contractor used a coarse gradation. When changing from a contractor's fine mix to another fine mix, the Bailey predicted VMA moved in the correct direction and provided reasonable values. That being noted, it would not be recommended, based on the discrepancies in the results of this thesis, that the Bailey Method be used to predict the change in VMA of a mixture when the gradation changes from coarse to fine or fine to coarse.

Table 21: Bailey predicted VMA versus actual VMA

Supplier	Mix Type	Gradation	Pill VMA	Bailey Predicted VMA	Difference
Greer	9.5 mm	Contractor (Fine)	16.6	-	-
Greer	9.5 mm	Coarse	17.0	15.1	1.9
Greer	9.5 mm	Fine	17.0	17.3	-0.3
Greer	12.5 mm	Contractor (Coarse)	16.8	-	-
Greer	12.5 mm	Coarse	17.6	16.5	1.1
Greer	12.5 mm	Fine	16.2	18.1	-1.9
Jefferson	9.5 mm	Contractor (Fine)	15.9	-	-
Jefferson	9.5 mm	Coarse	16.7	15.6	1.2
Jefferson	9.5 mm	Fine	17.1	16.7	0.4
Jefferson	12.5 mm	Contractor (Fine)	14.6	-	-
Jefferson	12.5 mm	Coarse	15.7	12.7	3.0
Jefferson	12.5 mm	Fine	15.3	14.8	0.5

The Bailey Method also provides guidance on the predicted constructability of a mix based on the aggregate ratios. Table 22 shows the aggregate ratios for each mix. Every effort was made to use gradations that satisfied the recommended ranges for each ratio, however, none of the mixes for Greer were able to satisfy all the requirements. It is possible that if another stockpile were added to the blend these ratio requirements could be satisfied. Each of the Greer mixes and the Jefferson 12.5 mm contractors mix had CA ratios above the recommended range which indicates potential for a tender mix.

Table 22: Bailey recommendations based on aggregate ratios

Supplier	Mix Type	Gradation	IDT Strength (kN/m^2)	CA	FA_c	FA_f	CA_{new}	$FA_{c,new}$	$FA_{f,new}$
Greer	9.5 mm	Contractor (Fine)	170	0.72	0.42	0.5	0.69	0.5	NA
Greer	9.5 mm	Coarse	149	0.55	0.41	0.54			
Greer	9.5 mm	Fine	208	2.07			0.6	0.45	NA
Greer	12.5 mm	Contractor (Coarse)	153	1.09	0.42	0.54	0.64	0.54	NA
Greer	12.5 mm	Coarse	143	0.95	0.39	0.55			
Greer	12.5 mm	Fine	185	1.57			0.6	0.47	NA
Jefferson	9.5 mm	Contractor (Fine)	221	0.54	0.39	0.39	0.65	0.39	NA
Jefferson	9.5 mm	Coarse	132	0.4	0.38	0.38			
Jefferson	9.5 mm	Fine	196	0.5			0.71	0.36	NA
Jefferson	12.5 mm	Contractor (Fine)	208	0.65	0.39	0.41	0.69	0.41	NA
Jefferson	12.5 mm	Coarse	149	0.55	0.41	0.46			
Jefferson	12.5 mm	Fine	217	0.64			0.7	0.4	NA

Note: The shaded cells represent a value that falls outside the recommended range indicating potential for a tender mix.

The IDT strength was plotted against the CA ratio for each supplier to observe the trend and is shown in Figure 14 and Figure 15. There is a clear positive relationship between the two. As the CA ratio increases, the IDT strength increases as well. This indicates that although the CA ratio is out of range for constructability, the pavement performance can be improved by a higher CA ratio due to a stronger aggregate structure.

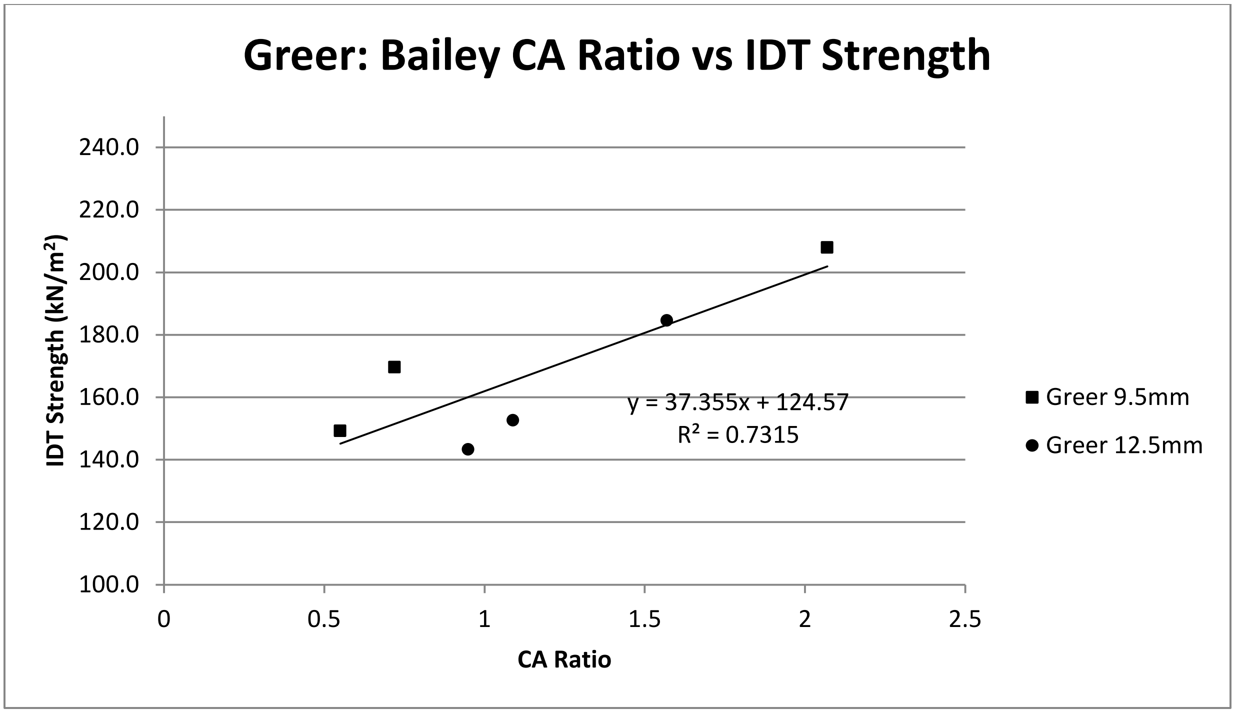

Figure 14: Greer Bailey CA ratio values versus IDT strength

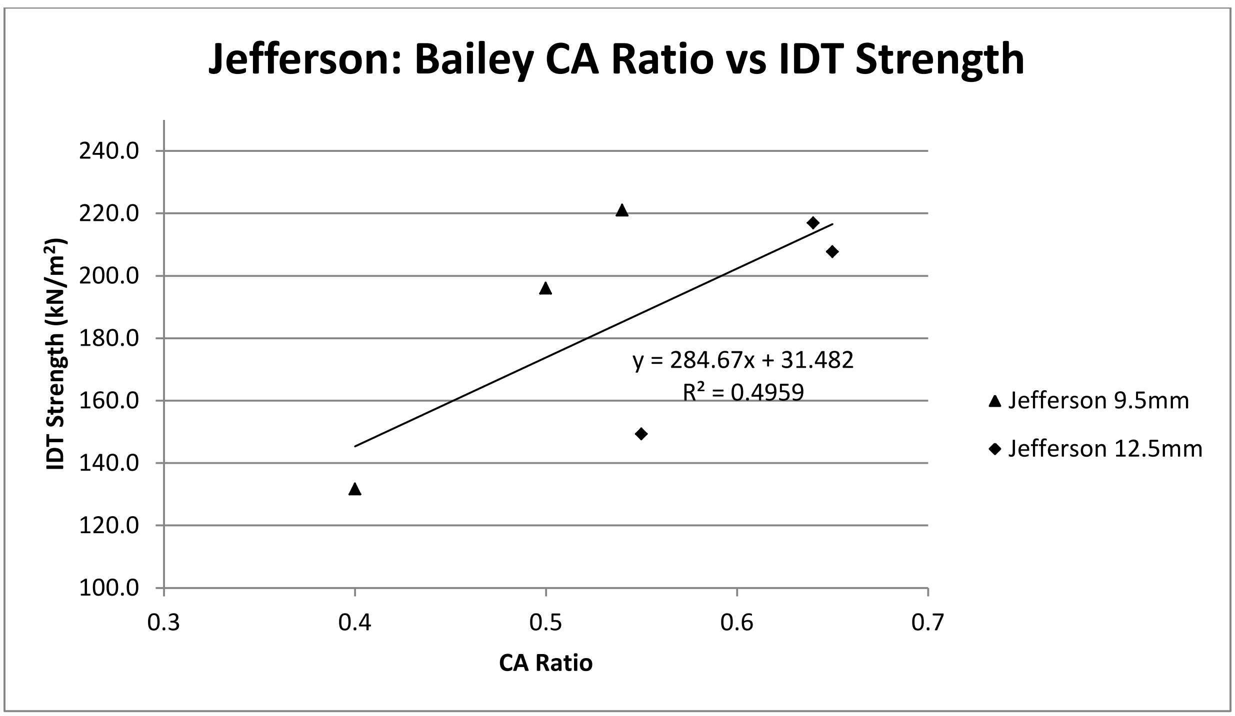

Figure 15: Jefferson Bailey CA ratio values versus IDT strength

The IDT strength of each mix was also compared to the CUW picked for each mix design. The results of this analysis are shown in Table 23 and graphically in Figure 16. As CUW increases, IDT decreases, which per Equation 11 suggests higher rutting potential.

Table 23: IDT strength versus chosen unit weight, CUW

Supplier	Mix Type	Gradation	IDT Strength (kN/m^2)	CUW
Greer	9.5 mm	Contractor (Fine)	170	81
Greer	9.5 mm	Coarse	149	95
Greer	9.5 mm	Fine	208	50
Greer	12.5 mm	Contractor (Coarse)	153	98
Greer	12.5 mm	Coarse	143	105
Greer	12.5 mm	Fine	185	75
Jefferson	9.5 mm	Contractor (Fine)	221	72
Jefferson	9.5 mm	Coarse	132	98
Jefferson	9.5 mm	Fine	196	50
Jefferson	12.5 mm	Contractor (Fine)	208	80
Jefferson	12.5 mm	Coarse	149	105
Jefferson	12.5 mm	Fine	217	65

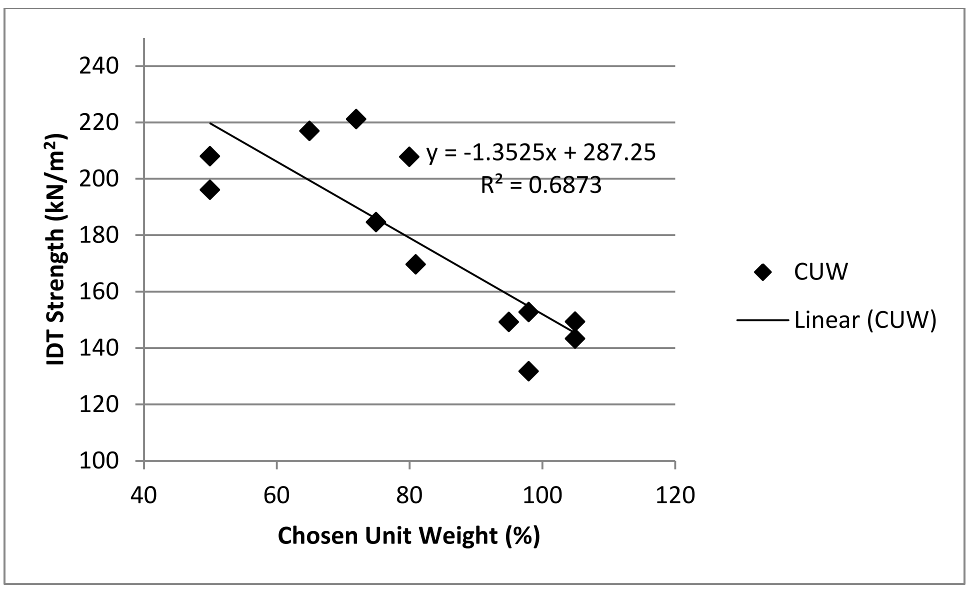

Figure 16: IDT strength versus CUW

4.3.5 Estimates of Effective Binder Volume

The volume of effective binder, V_{be}, determined from the pills was compared to the Superpave equation for V_{be} when determining initial binder content for the design aggregate structure, DAS, as well as and estimation based on the dry density samples. All values were converted to percent volume of binder effective in the mix. The only variable in the Superpave equation is the nominal maximum aggregate size, NMAS. Therefore, if the NMAS is 9.5 mm then the V_{be} would be estimated to be the same regardless of the gradation. VMA of a mix is equal to the V_{be} plus the air voids. Therefore, it is expected that the V_{be} change as the gradation changes for a specific NMAS as the VMA would likely change. The data presented in Table 24, as well as, Figure 17 and Figure 18 show that V_{be} changes as the gradation changes. The results from t-tests comparing the V_{be} of the pills to the Superpave and dry density estimates of V_{be} indicated that the null hypothesis of equal means is rejected with P values much less than 0.05. A linear regression indicated that based on the coefficients a and b, there is insufficient evidence to reject the null hypothesis of 1 and 0 respectively for the Superpave estimation while the dry density coefficients could be rejected with values much less than 0.05. This led to the determination that the estimation for V_{be} based on the dry density samples is less accurate than the Superpave DAS equation. However, the variability between the different methods is too great to provide useful relationships.

Table 24: Calculated V_{be} in pills versus estimates of needed V_{be} (per cm^3)

Supplier	Mix Type	Gradation	VTM	VMA	Actual V_{be} in pills	Superpave DAS Equation for V_{be}	Dry Density Estimation of V_{be}
Greer	9.5 mm	Contractor (Bailey Fine)	3.3	16.6	13.3	11.0	10.2
Greer	9.5 mm	Coarse	3.9	17.0	13.1	11.0	10.3
Greer	9.5 mm	Fine	3.9	17.0	13.1	11.0	10.1
Greer	12.5 mm	Contractor (Bailey Coarse)	4.0	16.8	12.7	10.2	10.7
Greer	12.5 mm	Coarse	4.9	17.6	12.7	10.2	10.8
Greer	12.5 mm	Fine	3.6	16.2	12.6	10.2	9.7
Jefferson	9.5 mm	Contractor (Bailey Fine)	4.3	15.9	11.6	11.0	9.6
Jefferson	9.5 mm	Coarse	3.9	16.7	12.8	11.0	10.8
Jefferson	9.5 mm	Fine	3.2	17.1	13.9	11.0	9.1
Jefferson	12.5 mm	Contractor (Bailey Fine)	3.5	14.6	11.1	10.2	9.8
Jefferson	12.5 mm	Coarse	3.0	15.7	12.7	10.2	11.5
Jefferson	12.5 mm	Fine	3.6	15.3	11.7	10.2	8.6

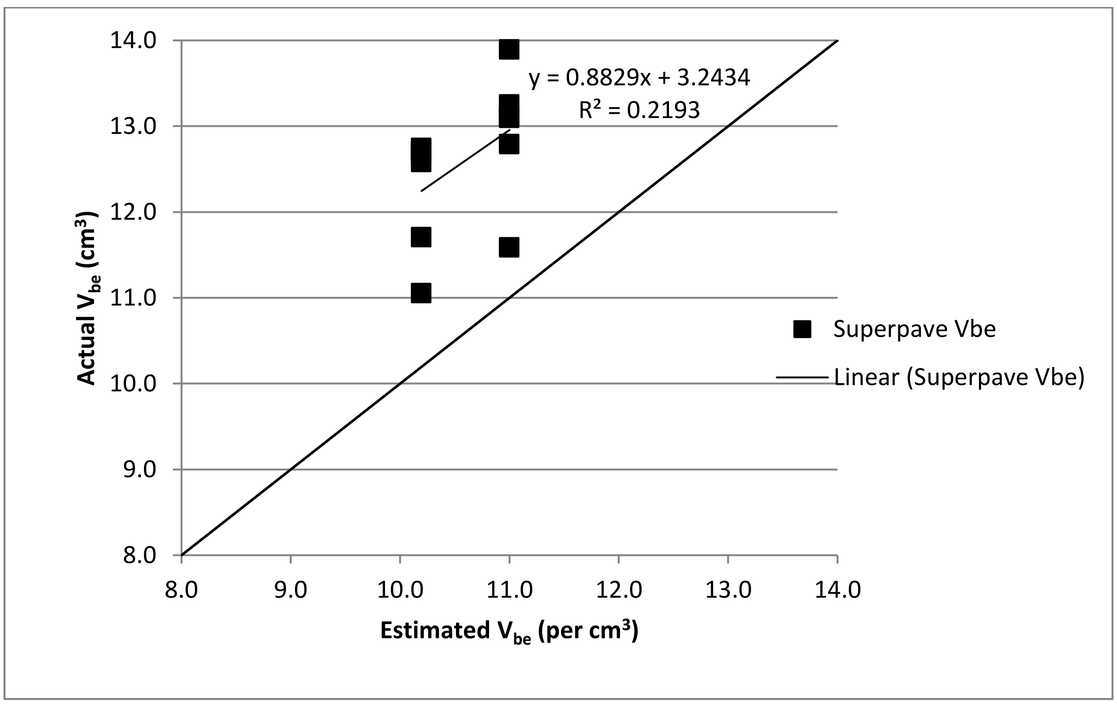

Figure 17: Actual V_{be} in pills versus Superpave estimations of V_{be}

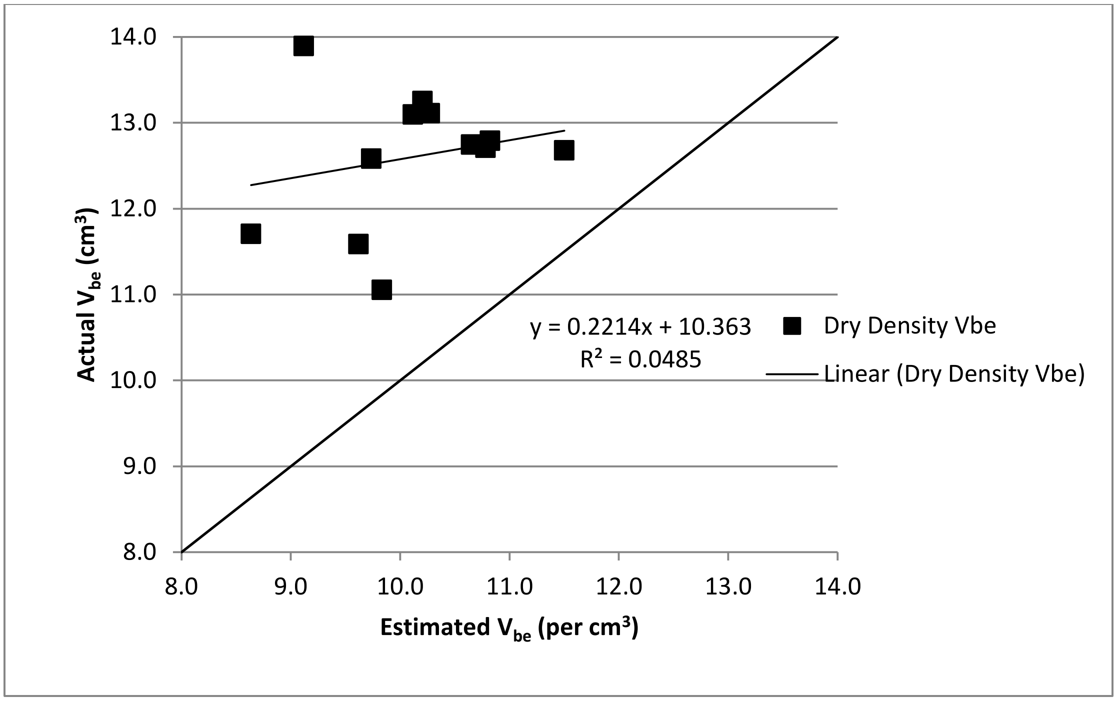

Figure 18: Actual V_{be} in pills versus dry density estimations of V_{be}

Chapter 5 CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

The objective of this thesis was to evaluate the use of aggregate density in asphalt mix design and determine the interaction with density and locking point. This objective was accomplished and all results were documented and statistically analyzed for support. Aggregate density in the asphalt mix is of importance because a strong aggregate structure is what carries the traffic load. The locking point was originally defined by Vavrik (2000) as the point where the aggregate is in its densest configuration and further compaction of the specimen would result in degradation of the aggregate. The Bailey Method provides guidance in selecting an aggregate gradation based on unit weights that promotes proper aggregate packing principles and formation of a strong aggregate structure.

The aggregate in the compacted asphalt samples was determined to be in a dense configuration based on the comparison with the maximum dry density. Aggregate density in the compacted samples averaged approximately 2 percent higher than the dry density. This lead to the determination that the aggregate had formed a dense configuration in the compactor and the asphalt was actually acting as a lubricant allowing the rocks to slide past each other and pack even tighter. With this being confirmed the locking point of the mixtures was evaluated next.

The locking points for each mix were evaluated based on multiple definitions; however, the original definition, 2-2-3, and the 2-2 locking point were the main focus. The results of this analysis show that for the Greer mixes, the 2-2-3 locking points were much higher than the design gyration level of 80, while the 2-2 locking point was very close to N_{des}. The Jefferson mixes, with the exception of the two coarse mixes, had2-2-3 locking points much closer, if not below 80 gyrations, while the 2-2 locking point was much less than N_{des}. This indicates that, based on the work of Li and Gibson (2011), the Greer mixes have achieved optimum density when compacted to N_{des} of 80 gyrations, while the Jefferson mixes could potentially be over-compacting at 80 gyrations and degrading the aggregate in the mix.

The VMA of each mix was compared to a dry VMA calculated from the samples in the dry density test. It was obvious from the results that there was a significant difference between the VMA of the compacted samples and the dry VMA due to the surface voids being included in the calculation for dry VMA. This led to a volume based VMA calculation for the pills in order to compare like values between the two. This calculation adjustment gave values that appeared more similar; and there was insufficient evidence to reject the null hypothesis of a t-test or the linear regression of the coefficients. However, due to the variability of the results it was determined that the pill VMA could not be reliably predicted from the dry VMA.

The Bailey Method was reviewed for its ability to provide accurate predictions of the asphalt mix performance based on the aggregate ratios. Results showed that even though many of the mixes had aggregate ratios outside the recommended range, they performed well in terms of IDT strength. All mixes tested for this thesis had IDT strengths much higher than the 12 psi recommended by Zaniewski and Srinivasan (2003) for adequate resistance to rutting. The Bailey defined fine mixes (including the contractors' mixes with Bailey defined fine gradations) demonstrated a higher IDT strength than the coarse mixes indicating that the fine mixes would have better rutting resistance overall than the coarse mixes. The relationship between the IDT strength and CA ratio was also evaluated and showed an increasing trend in the IDT strength as the CA ratio increases. In all cases in this thesis where the CA ratio failed, including all Greer mixes and the Jefferson 12.5 mm contractor's mix, it was above the range recommended by the

Bailey Method. This increase in IDT strength with increase in CA ratio indicated that while there may be potential constructability issues, the mix performance is improved with higher CA ratio.

The results of the actual pill V_{be} compared to the V_{be} predicted from the Superpave DAS equation and the dry density samples indicated that there was a significant difference in the values obtained from each. It was determined that the dry density samples were a poor predictor of the actual V_{be} as the results of both the t-test of the means and the t-test of the linear regression coefficients rejected the null hypothesis.

5.2 Recommendations

The research presented in this thesis evaluated the aggregate density in asphalt mixtures and the use of density in determining a design aggregate structure. General trends, supported by statistical analysis, were observed in the relationships of various parameters of the asphalt mixtures. Due to the issue of changing from a coarse to a fine mix or fine to a coarse mix in predicting the change in VMA of a mix, it was determined that using the Bailey for choosing a DAS was not recommended by this thesis. The trend that the IDT strength increased with an increase in CA ratio could be valuable in predicting performance of the pavement, however, the CA ratio is based on the gradation curve and could be determined without going through the entire Bailey Method process.

The size of this experiment limited the ability to definitively say that these inconsistencies prove the methods to be inaccurate. Further research is needed and therefore an expanded experiment focusing on each specific topic with more variations in asphalt suppliers, mix type, gradation changes and asphalt binder type could help to support the concepts developed in this thesis.

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APPENDIX

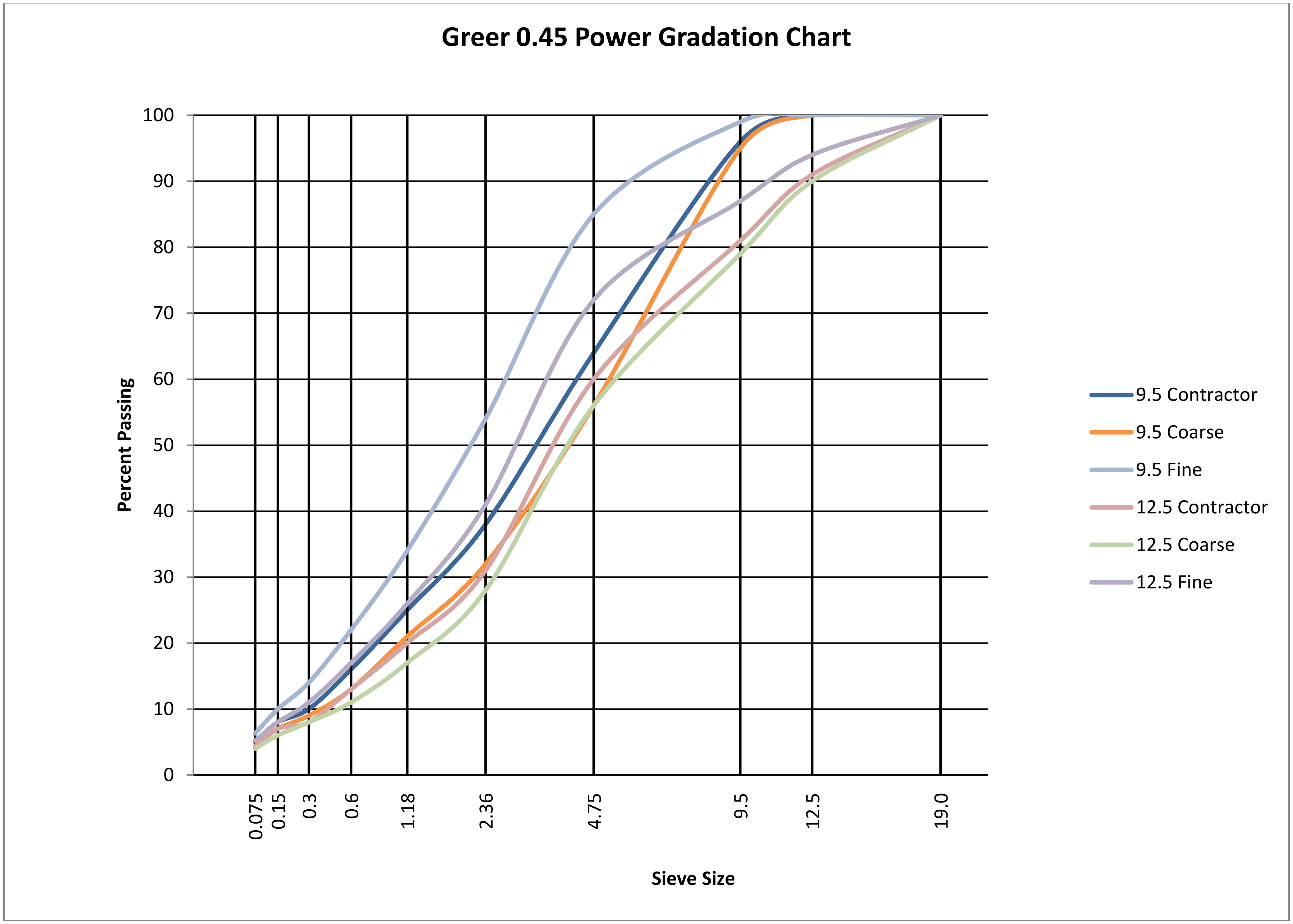

Figure 19: Greer gradations

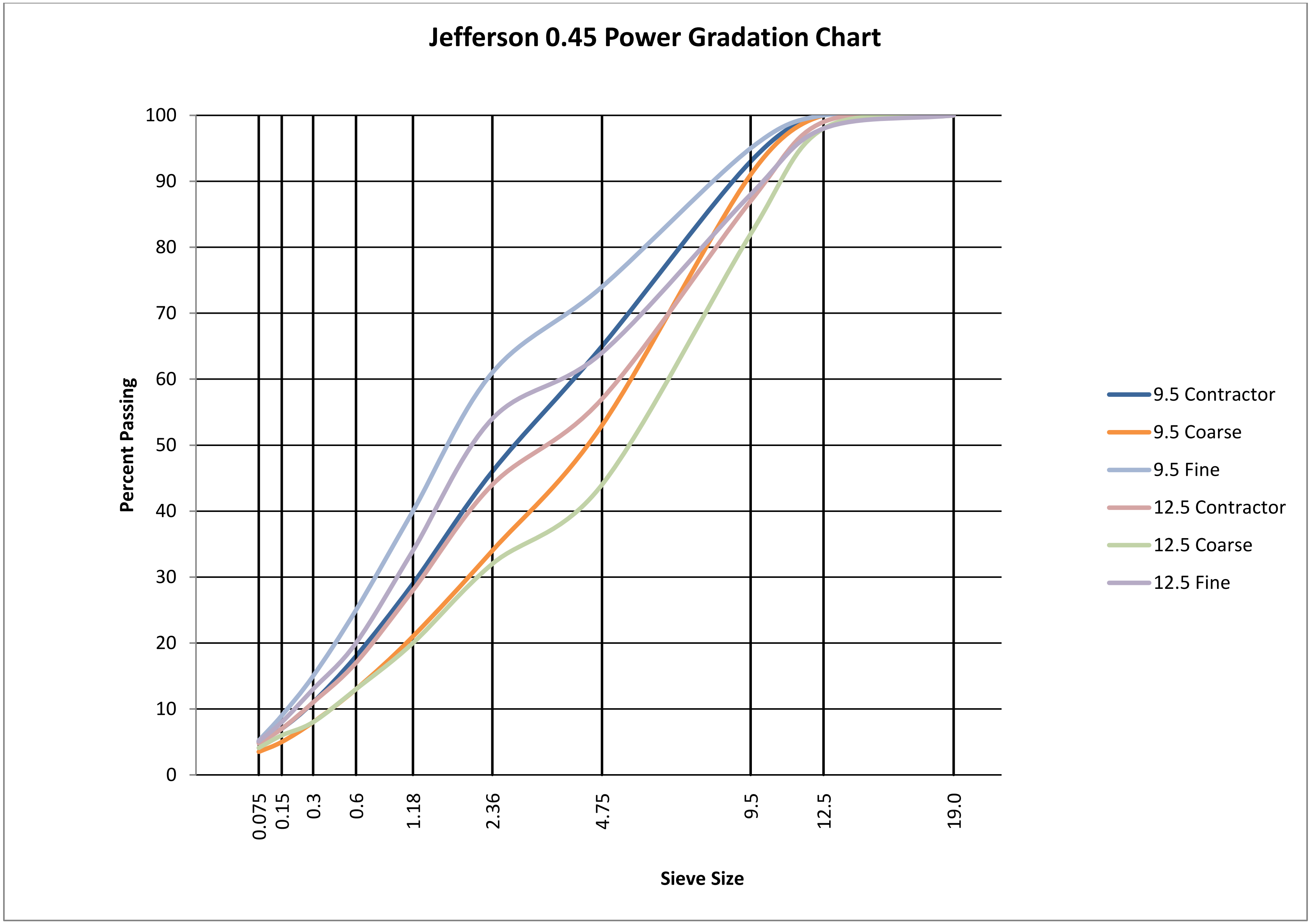

Figure 20: Jefferson gradations

Table 25: Individual pill results for Greer mixes

	Greer																	
	9.5 mm									12.5 mm								
	Contractor			Coarse			Fine			Contractor			Coarse			Fine		
Sample #	**4**	**5**	**6**	**10**	**11**	**12**	**16**	**17**	**18**	**22**	**23**	**24**	**28**	**29**	**30**	**34**	**35**	**36**
P_b	0.062	0.062	0.062	0.062	0.062	0.062	0.064	0.064	0.064	0.059	0.059	0.059	0.061	0.061	0.061	0.061	0.061	0.061
G_{mm}	2.447	2.447	2.447	2.455	2.455	2.455	2.439	2.439	2.439	2.456	2.456	2.456	2.462	2.462	2.462	2.458	2.458	2.458
G_{mb}	2.360	2.373	2.363	2.364	2.366	2.346	2.340	2.342	2.346	2.362	2.367	2.342	2.345	2.342	2.341	2.368	2.368	2.375
G_{sb}	2.660	2.660	2.660	2.667	2.667	2.667	2.643	2.643	2.643	2.665	2.665	2.665	2.668	2.668	2.668	2.655	2.655	2.655
VTM	3.6	3.0	3.4	3.7	3.6	4.4	4.0	4.0	3.8	3.8	3.6	4.6	4.8	4.9	4.9	3.7	3.7	3.4
VMA	16.8	16.3	16.7	16.9	16.8	17.5	17.1	17.0	16.9	16.6	16.4	17.3	17.5	17.6	17.6	16.3	16.2	16.0
Bulk VMA	21.1	20.7	21.0	21.4	21.4	22.1	21.0	20.9	21.0	21.3	20.9	22.0	22.2	22.4	22.4	20.5	20.5	20.3
H (m)	0.1158	0.1151	0.1157	0.1164	0.1164	0.1168	0.1169	0.1165	0.1162	0.1173	0.1169	0.1183	0.1184	0.1190	0.1185	0.1172	0.1170	0.1162
V (m^3)	0.0020	0.0020	0.0020	0.0021	0.0021	0.0021	0.0021	0.0021	0.0021	0.0021	0.0021	0.0021	0.0021	0.0021	0.0021	0.0021	0.0021	0.0021
Density (kg/m^3)	2165.6	2176.8	2168.5	2164.6	2165.1	2144.4	2155.9	2157.0	2156.4	2164.7	2175.7	2145.9	2142.2	2135.8	2136.5	2177.5	2179.1	2183.2
Stability (lbs)	1000	1088	1025	1025	813	925	1275	1275	1300	925	925	1000	913	863	925	1125	1150	1150
IDT Strength (psi)	23.6	25.9	24.3	24.1	19.1	21.7	29.9	30.0	30.6	21.6	21.7	23.1	21.1	19.8	21.4	26.3	26.9	27.1
Dry Mass	4724.6	4720.2	4726.8	4746.7	4747.8	4718.6	4758.1	4744.2	4730.8	4768.4	4776.3	4767.3	4773.4	4783.2	4764.7	4802.7	4798.1	4774.2
Sub. Mass	2730.7	2738.0	2734.2	2746.6	2750.5	2724.1	2732.3	2726.1	2719.6	2757.7	2766.8	2748.4	2751.4	2755.2	2744.2	2781.9	2778.8	2771.5
SSD Mass	4732.8	4726.9	4734.5	4754.4	4757.1	4735.1	4765.7	4751.5	4736.1	4776.9	4784.4	4784.0	4787.3	4797.9	4779.5	4810.3	4804.8	4781.9

Table 26: Individual pill results for Jefferson mixes

	Jefferson																	
	9.5 mm									12.5 mm								
	Contractor			Coarse			Fine			Contractor			Coarse			Fine		
Sample #	**40**	**41**	**42**	**46**	**47**	**48**	**52**	**53**	**54**	**58**	**59**	**60**	**64**	**65**	**66**	**70**	**71**	**72**
P_b	0.055	0.055	0.055	0.060	0.060	0.060	0.063	0.063	0.063	0.051	0.051	0.051	0.054	0.054	0.054	0.053	0.053	0.053
G_{mm}	2.581	2.581	2.581	2.583	2.583	2.583	2.570	2.570	2.570	2.645	2.645	2.645	2.614	2.614	2.614	2.619	2.619	2.619
G_{mb}	2.471	2.453	2.486	2.484	2.474	2.484	2.492	2.485	2.487	2.559	2.546	2.549	2.537	2.536	2.535	2.522	2.536	2.512
G_{sb}	2.775	2.775	2.775	2.801	2.801	2.801	2.812	2.812	2.812	2.835	2.835	2.835	2.845	2.845	2.845	2.823	2.823	2.823
VTM	4.3	4.9	3.7	3.8	4.2	3.8	3.1	3.3	3.3	3.3	3.8	3.6	3.0	3.0	3.0	3.7	3.2	4.1
VMA	15.9	16.5	15.3	16.6	17.0	16.6	17.0	17.2	17.1	14.3	14.8	14.7	15.7	15.7	15.7	15.4	14.9	15.7
Bulk VMA	20.3	20.9	19.6	21.4	21.8	21.4	20.9	21.1	20.9	18.8	19.3	19.2	20.9	20.7	20.9	19.5	19.2	19.8
H (m)	0.1112	0.1128	0.1108	0.1135	0.1141	0.1130	0.1120	0.1124	0.1114	0.1113	0.1123	0.1127	0.1140	0.1138	0.1136	0.1134	0.1128	0.1144
V (m^3)	0.0020	0.0020	0.0020	0.0020	0.0020	0.0020	0.0020	0.0020	0.0020	0.0020	0.0020	0.0020	0.0020	0.0020	0.0020	0.0020	0.0020	0.0020
Density (kg/m^3)	2284.4	2266.4	2302.1	2271.4	2261.2	2273.7	2297.1	2291.1	2295.1	2377.0	2361.1	2365.5	2323.0	2327.6	2323.2	2346.6	2354.8	2336.0
Stability (lbs)	1300	1625	1000	775	788	813	1213	1250	1025	1263	1200	1238	925	938	838	1350	1338	1225
IDT Strength (psi)	32.0	39.4	24.7	18.7	18.9	19.7	29.6	30.5	25.2	31.1	29.3	30.1	22.2	22.6	20.2	32.6	32.5	29.3
Dry Mass	4750.3	4780.6	4769.8	4846.6	4850.2	4830.1	4852.2	4856.8	4821.9	4926.5	4937.4	4964.3	4947.0	4948.1	4930.0	4965.7	4956.7	4981.5
Sub. Mass	2839.3	2849.7	2857.6	2906.8	2903.4	2895.0	2908.0	2905.8	2887.0	3005.8	3003.5	3023.5	3010.5	3005.9	2994.0	3002.7	3006.7	3006.0
SSD Mass	4761.9	4798.3	4776.0	4857.6	4863.8	4839.3	4855.3	4859.9	4826.1	4931.1	4943.0	4971.0	4960.7	4957.3	4938.5	4971.3	4961.2	4989.0

Table 27: Individual dry density test results for Greer mixes

	Greer																	
	9.5 mm									12.5 mm								
	Contractor			Coarse			Fine			Contractor			Coarse			Fine		
Sample #	1	2	3	7	8	9	13	14	15	19	20	21	25	26	27	31	32	33
Sample height (in)	4.797	4.766	4.789	4.758	4.742	4.781	4.742	4.789	4.750	4.906	4.922	4.789	4.719	4.789	4.766	4.898	4.789	4.773
Sample height (m)	0.122	0.121	0.122	0.121	0.120	0.121	0.120	0.122	0.121	0.125	0.125	0.122	0.120	0.122	0.121	0.124	0.122	0.121
Sample raidius (m)	0.076	0.076	0.076	0.076	0.076	0.076	0.076	0.076	0.076	0.076	0.076	0.076	0.076	0.076	0.076	0.076	0.076	0.076
V (m^3)	0.0022	0.0022	0.0022	0.0022	0.0022	0.0022	0.0022	0.0022	0.0022	0.0023	0.0023	0.0022	0.0022	0.0022	0.0022	0.0023	0.0022	0.0022
Vibrated Wt. M + S (kg)	9.615	9.571	9.635	9.622	9.569	9.563	9.551	9.577	9.521	9.674	9.620	9.642	9.503	9.522	9.540	9.695	9.655	9.647
Mold Wt. (kg)	4.957	4.957	4.957	4.957	4.957	4.957	4.957	4.957	4.957	4.957	4.957	4.957	4.957	4.957	4.957	4.957	4.957	4.957
VMA	21.2	21.4	20.7	20.7	21.3	22.0	20.9	21.2	21.5	22.1	23.3	20.8	22.1	22.9	22.2	21.4	20.3	20.1
Dry Density (kg/m^3)	2095.8	2089.6	2108.2	2116.2	2099	2079.2	2090.8	2082.1	2073.8	2075	2044.8	2111.4	2079.3	2057.3	2075.6	2087.6	2117.2	2120.5

Table 28: Individual dry density test results for Jefferson mixes

	Jefferson																	
	9.5 mm									12.5 mm								
	Contractor			Coarse			Fine			Contractor			Coarse			Fine		
Sample #	37	38	39	43	44	45	49	50	51	55	56	57	61	62	63	67	68	69
Sample height (in)	4.828	4.711	4.844	4.875	4.867	4.805	4.875	4.8125	4.8047	4.8359	4.6719	4.8516	4.8359	4.8906	4.9531	4.7734	4.8203	4.8672
Sample height (m)	0.123	0.120	0.123	0.124	0.124	0.122	0.124	0.122	0.122	0.1228	0.1187	0.1232	0.1228	0.1242	0.1258	0.1212	0.1224	0.1236
Sample raidius (m)	0.076	0.076	0.076	0.076	0.076	0.076	0.076	0.076	0.076	0.0762	0.0762	0.0762	0.0762	0.0762	0.0762	0.0762	0.0762	0.0762
V (m^3)	0.0022	0.0022	0.0022	0.0023	0.0023	0.0022	0.0023	0.0022	0.0022	0.0022	0.0022	0.0022	0.0022	0.0023	0.0023	0.0022	0.0022	0.0023
Vibrated Wt. M + S (kg)	9.981	9.912	10.002	9.987	9.947	9.913	10.119	10.091	10.116	10.087	9.916	10.124	9.991	10.027	10.015	10.127	10.156	10.177
Mold Wt. (kg)	4.957	4.957	4.957	4.957	4.957	4.957	4.957	4.957	4.957	4.957	4.957	4.957	4.957	4.957	4.957	4.957	4.957	4.957
VMA	19.1	18.2	19.0	20.5	21.0	20.5	18.7	18.1	17.6	19.2	19.2	18.9	21.0	21.4	22.5	17.2	17.5	18.0
Dry Density (kg/m^3)	2245.8	2270.1	2247.9	2226.9	2212.7	2226.2	2285.3	2302.5	2317.4	2289.5	2290.9	2298.6	2246.7	2237.4	2204	2337.6	2327.8	2314.7

SUMMARY OUTPUT

Regression Statistics	
Multiple R	0.960
R Square	0.922
Adjusted R Square	0.915
Standard Error	24.749
Observations	12

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	1	72884.91	72884.91	119.00	7.12436E-07
Residual	10	6125.00	612.50		
Total	11	79009.91			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*
Intercept	459.303	163.172	2.815	0.018	95.733	822.873
X Variable 1	0.816	0.075	10.909	0.000	0.649	0.982
tails	2		Decision			
t for Hn = 1	-2.465		Intercept	reject Hn		
p-value for Hn = 1	0.033		Slope	reject Hn		

Figure 21: Linear regression of dry density versus aggregate density in pills

t-Test: Two-Sample Assuming Unequal Variances

	Dry Density	*Agg. Den. In Pills*
Mean	2180	2238
Variance	9959	7183
Observations	12	12
Hypothesized Mean Difference	0	
df	21	
t Stat	-1.519	
P(T<=t) one-tail	0.072	
t Critical one-tail	1.721	
P(T<=t) two-tail	0.144	
t Critical two-tail	2.080	

Figure 22: t-test of dry density versus aggregate density in pills

SUMMARY
OUTPUT

Regression Statistics	
Multiple R	0.959
R Square	0.920
Adjusted R Square	0.912
Standard Error	24.998
Observations	12

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	1	72262.238	72262.238	115.637	8.13646E-07
Residual	10	6249.048	624.905		
Total	11	78511.286			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*
Intercept	467.939	164.816	2.839	0.018	100.706	835.173
X Variable 1	0.812	0.076	10.753	0.000	0.644	0.980
tails	2		Decision			
t for Hn = 1	-2.487		Intercept	reject Hn		
p-value for Hn = 1	0.032		Slope	reject Hn		

Figure 23: Linear regression of dry density versus aggregate density in pills at LP 2-2

SUMMARY OUTPUT

Regression Statistics	
Multiple R	0.955
R Square	0.912
Adjusted R Square	0.903
Standard Error	26.030
Observations	12

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	1	70124.550	70124.550	103.492	1.35788E-06
Residual	10	6775.837	677.584		
Total	11	76900.387			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*
Intercept	511.810	171.623	2.982	0.014	129.411	894.209
X Variable 1	0.800	0.079	10.173	0.000	0.625	0.975
tails	2		Decision			
t for Hn = 1	-2.542		Intercept	reject Hn		
p-value for Hn = 1	0.029		Slope	reject Hn		

Figure 24: Linear regression of dry density versus aggregate density in pills at LP 2-2-3

t-Test: Two-Sample Assuming Unequal Variances

	Pill VMA	*Dry VMA*
Mean	16.4	20.4
Variance	0.7	2.5
Observations	12	12
Hypothesized Mean Difference	0	
df	17	
t Stat	-7.68	
P(T<=t) one-tail	3.14E-07	
t Critical one-tail	1.74	
P(T<=t) two-tail	6.28E-07	
t Critical two-tail	2.11	

Figure 25: t-test of pill VMA versus dry VMA

t-Test: Two-Sample Assuming Unequal Variances

	Volume VMA	*Dry VMA*
Mean	20.8	20.4
Variance	0.8	2.5
Observations	12	12
Hypothesized Mean Difference	0	
df	18	
t Stat	0.84	
P(T<=t) one-tail	0.21	
t Critical one-tail	1.73	
P(T<=t) two-tail	0.41	
t Critical two-tail	2.10	

Figure 26: t-test of volume VMA versus dry VMA

t-Test: Two-Sample Assuming Unequal Variances

	Pill VMA Change	Dry VMA Change
Mean	0.62	0.18
Variance	0.32	2.12
Observations	8	8
Hypothesized Mean Difference	0	
df	9	
t Stat	0.80	
P(T<=t) one-tail	0.22	
t Critical one-tail	1.83	
P(T<=t) two-tail	0.44	
t Critical two-tail	2.26	

Figure 27: t-test of VMA change in pills versus dry VMA change

t-Test: Two-Sample Assuming Equal Variances

	SP V_{be}	*Actual V_{be}*
Mean	10.59811154	12.6009353
Variance	0.176518235	0.627574207
Observations	12	12
Pooled Variance	0.402046221	
Hypothesized Mean Difference	0	
df	22	
t Stat	-7.737138446	
P(T<=t) one-tail	5.11639E-08	
t Critical one-tail	1.717144335	
P(T<=t) two-tail	1.02328E-07	
t Critical two-tail	2.073873058	

Figure 28: t-test of Superpave estimated V_{be} versus actual pill V_{be}

SUMMARY OUTPUT

Regression Statistics	
Multiple R	0.468
R Square	0.219
Adjusted R Square	0.141
Standard Error	0.734
Observations	12

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	1	1.514	1.514	2.809	0.125
Residual	10	5.390	0.539		
Total	11	6.903			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*
Intercept	3.243	5.588	0.580	0.574	-9.207	15.693
X Variable 1	0.883	0.527	1.676	0.125	-0.291	2.057
tails	2		Decision			
t for Hn = 1	-0.222		Intercept	cannot reject Hn		
p-value for Hn = 1	0.829		Slope	cannot reject Hn		

Figure 29: Linear regression of Superpave estimated V_{be} versus actual pill V_{be}

t-Test: Two-Sample Assuming Equal Variances

	Dry Density V_{be}	*Actual V_{be}*
Mean	10.10875226	12.6009353
Variance	0.62034358	0.627574207
Observations	12	12
Pooled Variance	0.623958894	
Hypothesized Mean Difference	0	
df	22	
t Stat	-7.728186094	
P(T<=t) one-tail	5.214E-08	
t Critical one-tail	1.717144335	
P(T<=t) two-tail	1.0428E-07	
t Critical two-tail	2.073873058	

Figure 30: t-test of dry density estimated V_{be} versus actual pill V_{be}

SUMMARY OUTPUT

Regression Statistics	
Multiple R	0.220
R Square	0.048
Adjusted R Square	-0.047
Standard Error	0.810
Observations	12

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	1	0.335	0.335	0.509	0.492
Residual	10	6.569	0.657		
Total	11	6.903			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*
Intercept	10.363	3.145	3.295	0.008	3.355	17.370
X Variable 1	0.221	0.310	0.714	0.492	-0.470	0.913

tails	2	Decision	
t for Hn = 1	-2.509	Intercept	reject Hn
p-value for Hn = 1	0.031	Slope	reject Hn

Figure 31: Linear regression of dry density estimated V_{be} versus actual pill V_{be}